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US Army Corps
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Waterways Experiment
Station

Flood Control Channels Research Program

Application of Channel Stability Methods — Case Studies

by Ronald R. Copeland

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Prepared for Headquarters, U.S. Army Corps of Engineers

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Application of Channel Stability Methods — Case Studies

by Ronald R. Copeland

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Waterways Experiment Station
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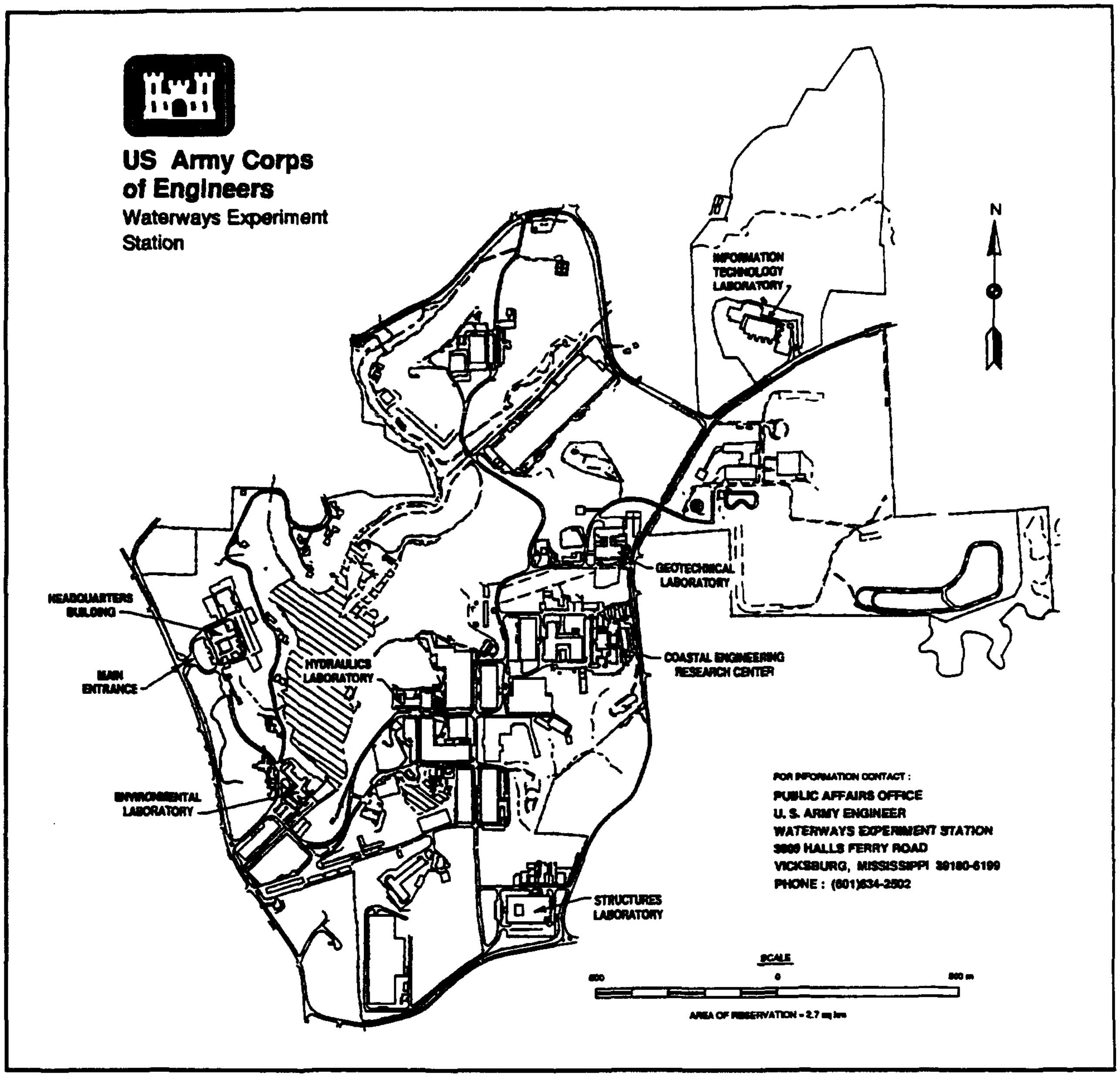

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Preface

The application of channel stability methods for two case studies, presented herein, is based on procedures outlined in Engineer Manual (EM) 1110-2-1418. The work was conducted at the U.S. Army Engineer Waterways Experiment Station (WES) as part of the Flood Control Channels Research Program, Work Unit 32549, "Controlling Stream Response to Channel Modification."

This investigation was conducted between July 1990 and April 1991 in the Hydraulics Laboratory (HL) of WES under the direction of Messrs. Frank A. Herrmann, Jr., Director, HL, WES; R. A. Sager, Assistant Director, HL; Marden B. Boyd, Chief, Waterways Division (WD); and Michael J. Trawle, Chief, Math Modeling Branch (MMB), WD. Mr. William A. Thomas, WD, was the project manager of the Flood Controls Channel Research Program. The project engineer and author of this report was Dr. Ronald R. Copeland, MMB.

The technical sponsor of the Flood Control Channels Research program was Mr. Thomas E. Munsey, Hydraulics and Hydrology Branch, Engineering Division, U.S. Army Corps of Engineers.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
miles (U.S. statute)	1.609347	kilometers
pounds	0.4536	kilograms
square miles	2.589998	square kilometers

1 Introduction

The case studies presented in this report were prepared in order to demonstrate the applicability of channel stability methods presented in Engineer Manual (EM) 1110-2-1418.[1] These methods are expected to have limited applicability for some streams and greater applicability in others. The two case studies chosen herein represent widely different stream types. Big and Colewa Creeks are located in a humid climate; they have mild slopes, a fine-sand bed, and cohesive banks. The Puerco River, an ephemeral stream located in an arid climate, has erodible banks and a fine-sand bed.

Methods presented in EM 1110-2-1418 are qualitative techniques intended for the planning and preliminary design stages of local flood-control projects. In the detailed design phase of flood-control projects, other techniques, including numerical and physical modeling, are currently used. However, such detailed methods are often not required for evaluating the feasibility of a flood-control channel project in general or for comparing the performance of alternative designs. The methods range from threshold and regime analyses appropriate for fixed-bed channels, t. analytical techniques appropriate for streams with significant sediment transport. All of these methods assume uniform flow.

[1] Headquarters, U.S. Army Corps of Engineers. (1994). "Channel stability assessment for flood control projects," Engineer Manual 1110-2-1418, Washington, DC.

2 Case Study, Big and Colewa Creeks, Louisiana

Introduction

Big and Colewa Creeks are approximately 75 miles[1] long and drain an area of about 550 square miles. The lower reach is known as Big Creek and the upper reach as Colewa Creek (Figure 1). It is a meandering low gradient stream in the Mississippi River Valley, originally draining forest and swamp land, but now surrounded by agricultural land. The stream has entrenched into alluvial deposits throughout its length. The surface of the floodplain is composed of clays and silts to a depth of about 20 ft, with sand and gravel below. Banks are generally vegetated with grass and shrubs and have localized bank erosion at bends (Figures 2 and 3). The banks are composed primarily of silt and clay. Limited bed sampling from 1981 to 1982 indicated that the bed contained both cohesive and non-cohesive materials. The samples were taken at four gauging stations. Generally, four lateral samples were taken at each site. At the upstream end of the study reach, at mile 72,[2] the bed was about 90 percent cohesive. At mile 53, the bed was 50 percent cohesive. Downstream at miles 40 and 18, the bed was 15 and 22 percent cohesive, respectively. Channel slopes generally increased in the downstream direction, a condition opposite to that found in most streams. This condition suggests a system in which channel capacity has historically decreased in the downstream direction and significant flow escaped onto the floodplain, or a system under natural or artificial base level control. The 1976 thalweg and bank profiles are shown in Figure 4.

To improve drainage, the Corps of Engineers, over a period of 40 years, applied various channel improvement techniques to various reaches of the river. These included clearing and snagging, channel enlargement, and cutoffs. Low-water weirs have been constructed in the channel (Figure 5). These weirs form permanent pools, which retard vegetative

[1] A table of factors for converting non-SI units of measurement to SI units is found on page vi.

[2] Creek miles are based on 1979 survey.

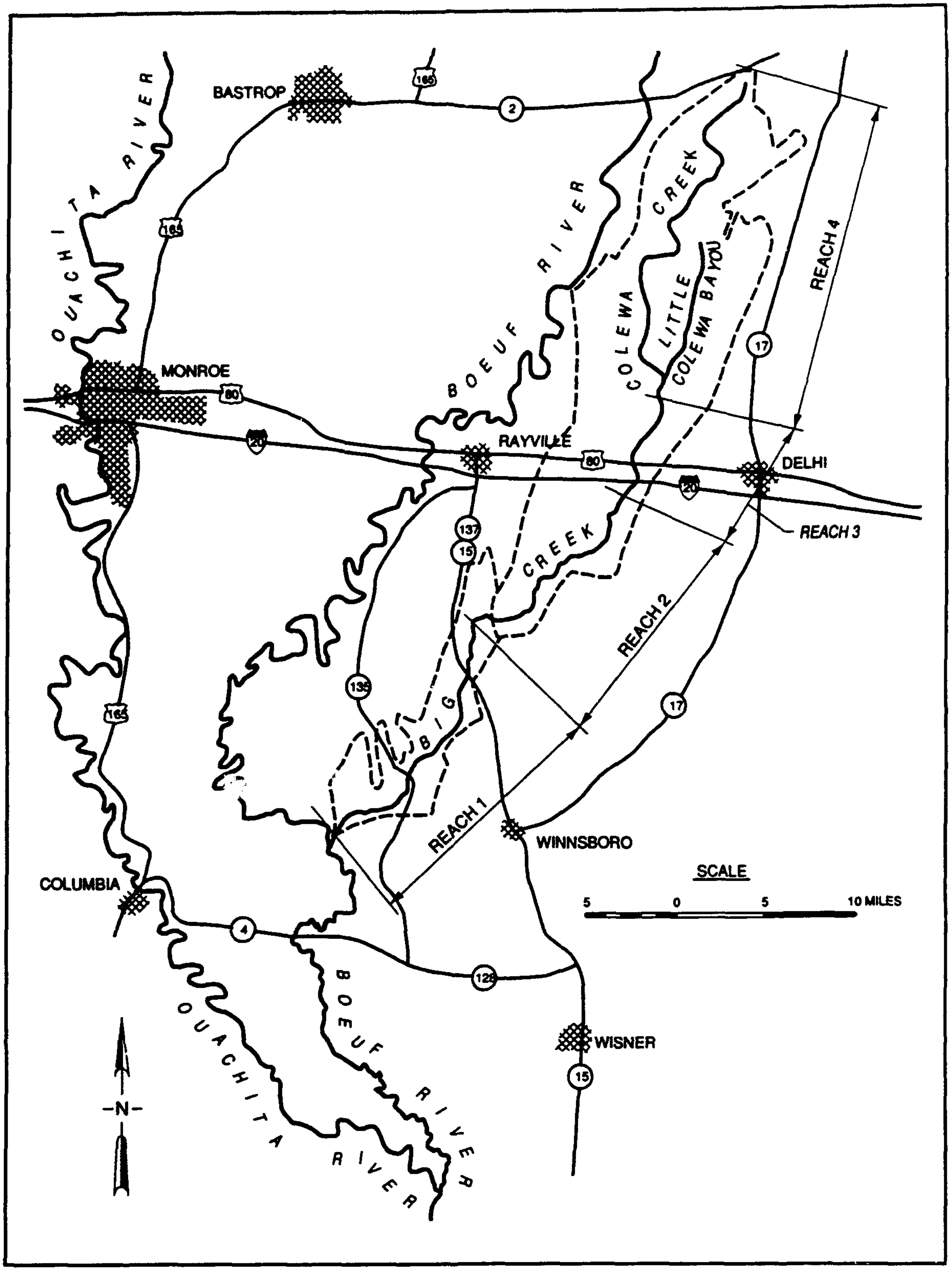

Figure 1. Location map - Big and Colewa Creeks

growth, thus reducing maintenance costs and improving hydraulic efficiency. The weirs also provide a measure of grade control. The first channel improvements by the Corps of Engineers were constructed from 1947 to 1954. The channel was designed to contain a 1- to 2-year return period flood. By 1959, it was determined that flood peaks had increased due to land use change and improved drainage. Some channel improvement work continued from 1954 to 1966, and then significant channel enlargements were constructed from 1966 to 1976 between miles 0 and 40.

The creek was divided into four reaches for the stability analysis. Reach 1 extends from the mouth of Big Creek to river mile 21. Reach 2 extends from river mile 21 to 38. Reach 3 extends from river mile 38 to 45, and reach 4 extends from river mile 45 to 72. Reach 4 is the upstream-most reach in the analysis and has the mildest slope. Prior to channel improvements, this reach was essentially a wide swampy swale, heavily vegetated with trees and brush.

Channel Stability

For the channel stability assessment, it was assumed that the channel was relatively stable in 1947, and that the purpose of the channel improvements was flood control. The 1947 to 1954 project included clearing and snagging in reach 1. This was accounted for in the hydraulic calculations by changing the bank roughness from 0.080 to 0.050. Channel cutoffs accompanied clearing and snagging in reach 2. This resulted in a decrease in bank roughness and an increase in channel slope. Improvements in reach 3 included channel excavation, cutoffs, and clearing and snagging. This resulted in a somewhat smaller cross-sectional area, reduced bank roughness, and increased slope. In reach 4, a channel was cut into what had previously had been a wide swampy swale. The average channel slope was unchanged in reach 4.

The second significant change to the system occurred when flood peaks increased due to channelization in the drainage basin. This was assumed to also change the channel-forming discharge. In this example, 1966 preproject conditions were assumed to have the same geometry as the 1954 design conditions, so that the effect of changing discharge could be assessed.

The third change to this system occurred with the 1966 to 1976 channel improvement project. This included channel enlargement and cutoffs in reaches 1 and 2. Changes in reaches 3 and 4 occurred naturally as the channel adjusted to previous improvement work.

Engineering documents do not mention stability problems prior to the 1966 to 1976 improvements, except for local scour and flanking of low-water weirs. Even after 1976, the stream appeared relatively stable. Problems included increased sediment transport through Big and Colewa

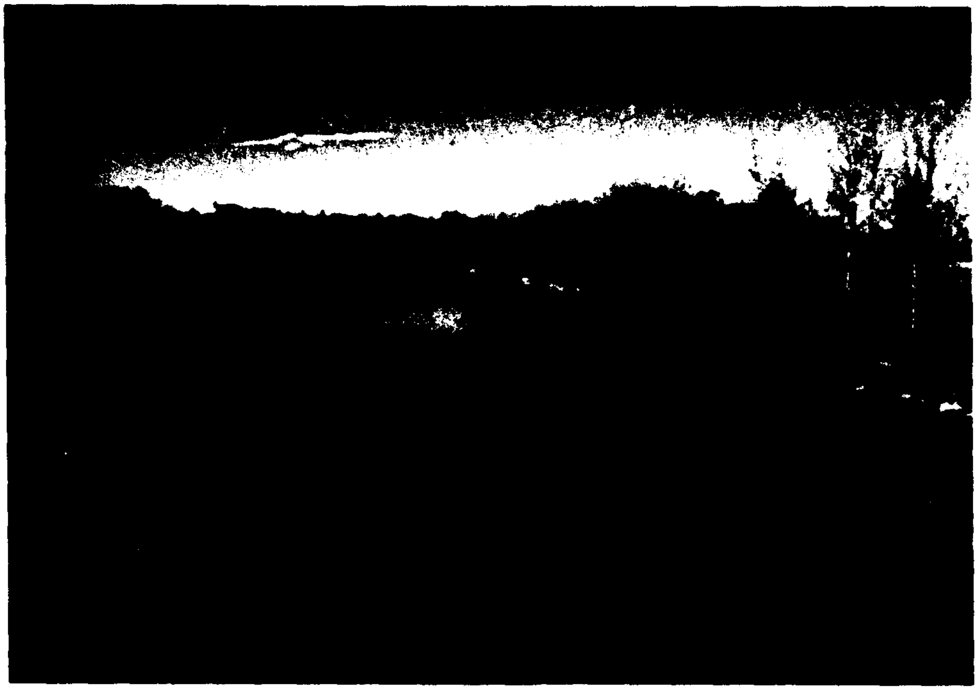

Figure 2. Upstream view of typical channel - river mile 20

Figure 3. Upstream view of localized bank erosion - river mile 31

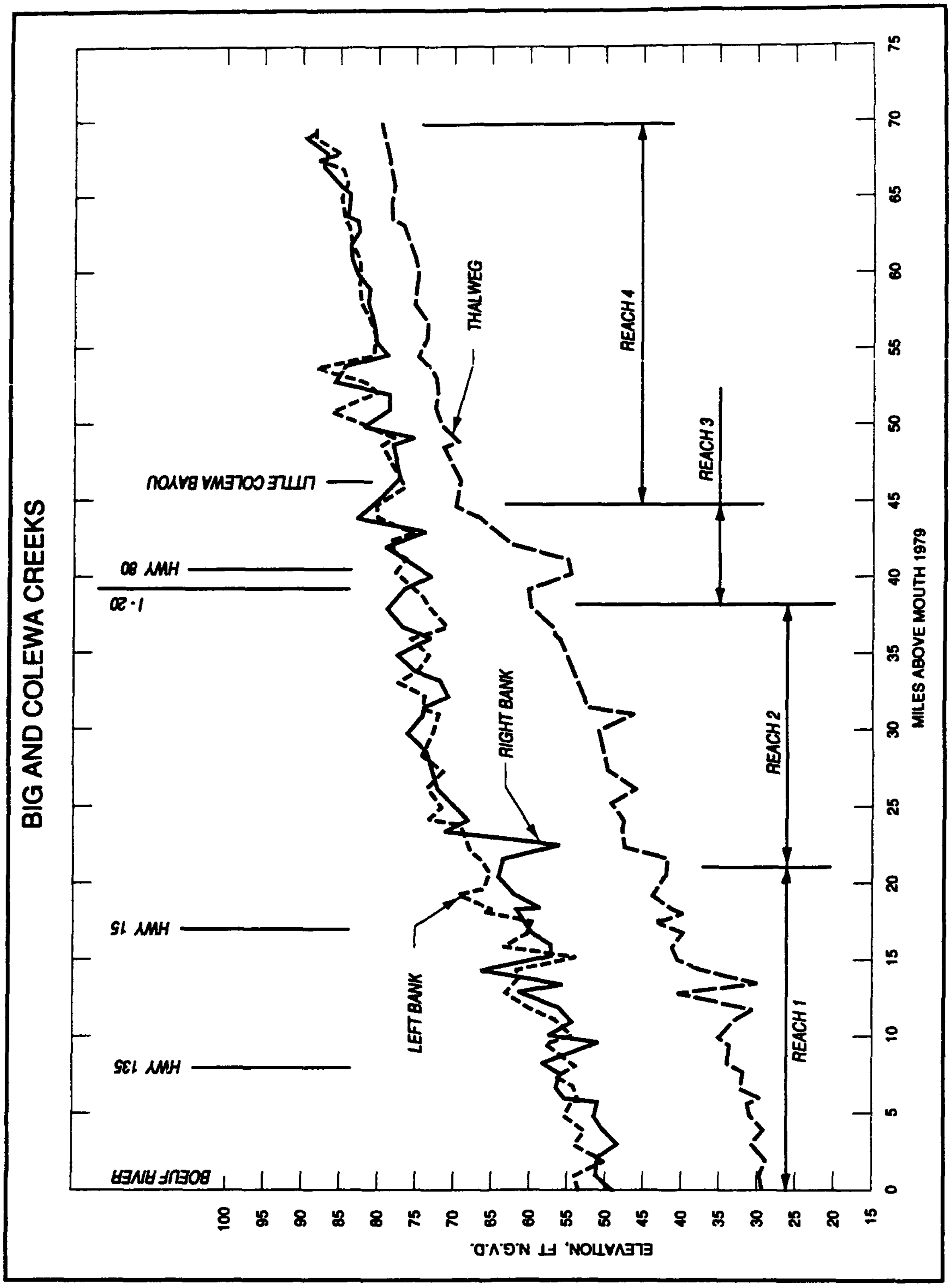

Figure 4. Thalweg and bank elevation profiles

Creeks so that deposition occurred downstream of the mouth of Big Creek in the Boeuf River. Overall, the stream appeared to be degrading, but sedimentation occurred at the upstream end of reach 2 (53,000 cu yd from 1976 to 1979). This filling was attributed to upstream degradation due to lowering of stages by the 1966 to 1976 project. In some cases the degradation moved up into tributary basins.

The first step in the stability assessment was to evaluate the stability of the channel in each of the four reaches, using the techniques outlined in EM 1110-2-1418. The channel-forming discharge was the design discharge which was considered to have a 1- to 2-year return period. This is reasonable for perennial streams in humid climates. Data needed to conduct the stability assessment included channel cross sections, channel slopes, and bed material samples.

Critical Velocity and Shear Stress

The critical velocity and shear stress methods were applied to determine if the channel exceeded threshold conditions for the channel-forming discharge. Hydraulic parameters were calculated for a typical cross section, using a normal depth algorithm. The typical cross sections were determined from channel survey data. Channel dimensions and assigned hydraulic parameters are listed in Table 1, and calculated hydraulic parameters are listed in Table 2. In this example, normal depth was calculated using The Hydraulic Design Package for Flood Control Channels (SAM).[1] Bank roughness was assigned, but bed roughness was calculated using the Brownlie equation. Hydraulic roughness due to channel irregularity, variations in cross-section shape, obstructions in the channel, and sinuosity, could only be accounted for in the assignment of bank roughness.

With a median grain diameter of 0.2 mm, EM 1110-2-1601[2] suggests a permissible mean velocity of 2 fps. EM 1110-2-1418 suggests approximate critical velocities of 2, 3, and 4.5 fps for flow depths of 5, 10, and 20 ft, respectively. Comparing these critical velocities to calculated velocities in Table 2, one would conclude that the 1947 preproject channel was stable in all reaches. With channel improvements, critical velocities were eventually exceeded according to the EM 1110-2-1601 criteria but were not exceeded according to the EM 1110-2-1418 criteria. It is known from the prototype that sediment movement occurred in the 1976 design channel. Thus, the EM 1110-2-1418 criteria cannot be applied to determine threshold conditions but may be appropriate in terms of channel stability.

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1 Thomas, W. A., Copeland, R. R., Raphelt, N. K., and McComas, D. N. (1994). "Hydraulic Design Package for Channels, SAM," U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

2 Headquarters, U.S. Army Corps of Engineers. "Hydraulic design of flood control channels," Engineer Manual 1110-2-1601, Washington, DC.

a. River mile 59

b. River mile 31

Figure 5. Low-water weirs

Table 1
Assigned Hydraulic Parameters, Channel Forming Discharge, Big and Colewa Creeks, Louisiana

Reach No.	Base Width, ft	Side Slope	Bed Slope	Bank, n	Discharge, cfs
1947 Preproject Conditions					
1	65	3H/1V	0.00013	0.080	5300
2	80	3H/1V	0.00013	0.080	3600
3	85	7H/1V	0.00019	0.080	2800
4	260	8H/1V	0.00009	0.080	2500
1954 Design Conditions					
1	65	3H/1V	0.00013	0.050	5300
2	80	3H/1V	0.00016	0.050	3600
3	70	3H/1V	0.00022	0.050	2800
4	80	2H/1V	0.00009	0.050	2500
1966 Preproject Conditions					
1	65	3H/1V	0.00013	0.050	9100
2	80	3H/1V	0.00016	0.050	5500
3	70	3H/1V	0.00022	0.050	4600
4	80	2H/1V	0.00009	0.050	4200
1976 Design Conditions					
1	140	3H/1V	0.00017	0.050	9100
2	110	3H/1V	0.00018	0.050	5500
3	53	3.5H/1V	0.00022	0.050	4600
4	63	3.2H/1V	0.00011	0.050	4200

Table 2
Calculated Hydraulic Parameters, Channel Forming Discharge, Big and Colewa Creeks, Louisiana

Reach No.	Top Width ft	Composite n	Velocity fps	Depth ft	Shield's Parameter	Conc. ppm
1947 Preproject Conditions						
1	210	0.066	1.6	24.2	2.92	19
2	187	0.060	1.5	17.8	2.15	22
3	260	0.064	1.3	12.5	2.22	18
4	375	0.043	1.1	7.2	0.60	7
1954 Design Conditions						
1	181	0.042	2.2	19.4	2.34	47
2	160	0.038	2.2	13.4	1.98	74
3	138	0.038	2.4	11.4	2.33	115
4	131	0.035	1.8	12.8	1.07	30
1966 Preproject Conditions						
1	219	0.043	2.5	25.7	3.10	51
2	183	0.040	2.5	17.1	2.54	82
3	161	0.040	2.6	15.1	3.08	128
4	151	0.038	2.1	17.7	1.48	36
1976 Design Conditions						
1	240	0.037	2.9	16.6	2.62	144
2	193	0.037	2.6	13.9	2.32	124
3	170	0.043	2.5	16.7	3.41	89
4	178	0.042	1.9	18.0	1.83	29

Note: $d_{50} = 0.000656$ ft, $\theta = 927$ DS.

To use the critical shear stress method, dimensionless shear stress θ is calculated from the following equation:

$$\theta = \frac{\gamma D S}{(\gamma_s - \gamma)\, d_{50}} \tag{1}$$

where

θ = dimensionless shear stress or Shield's parameter

γ = specific weight of water

D = water depth

S = Slope

γ_s = specific weight of sediment

d_{50} = Median grain size of bed material

Dimensionless shear stresses for the four conditions are tabulated in Table 2, and all are significantly greater than the critical value of 0.090 recommended in EM 1110-2-1418 (Figures 2, 3, and 4).

Using criteria from EM 1110-2-1418, critical velocity analyses indicated a stable channel, but the critical shear stress analyses suggested instability. Based on prototype experience, one would be inclined to discount the critical shear stress approach. The shear stress method is apparently more applicable to coarse bed streams.

Hydraulic Geometry

Average hydraulic parameters of slope, top width, and depth for preproject and design conditions are tabulated in Table 2. *Slope* is average bed slope in the reach. Top width and depth are calculated using the channel-forming discharge. *Depth* is the maximum depth in the trapezoidal cross section.

Data for 1947 to 1954 conditions are plotted in Figure 6; and 1966 to 1976 conditions are in Figure 7. In its 1947 condition, reach 4 was not really a channel, but a swampy swale, and would not be expected to match well with the hydraulic geometry charts. The banks of Big and Colewa Creeks are cohesive and would be expected to fall between curves 1 and 2 on the width prediction chart. However, top widths for all conditions plotted on or above curve 2. Thus, prototype top widths were greater than predicted using the hydraulic geometry charts.

Using a median grain size of 0.2 mm, the depth and slope could be predicted from the hydraulic geometry charts. Prototype depths were greater than predicted from the charts for both 1947 to 1954 and 1966 to 1976

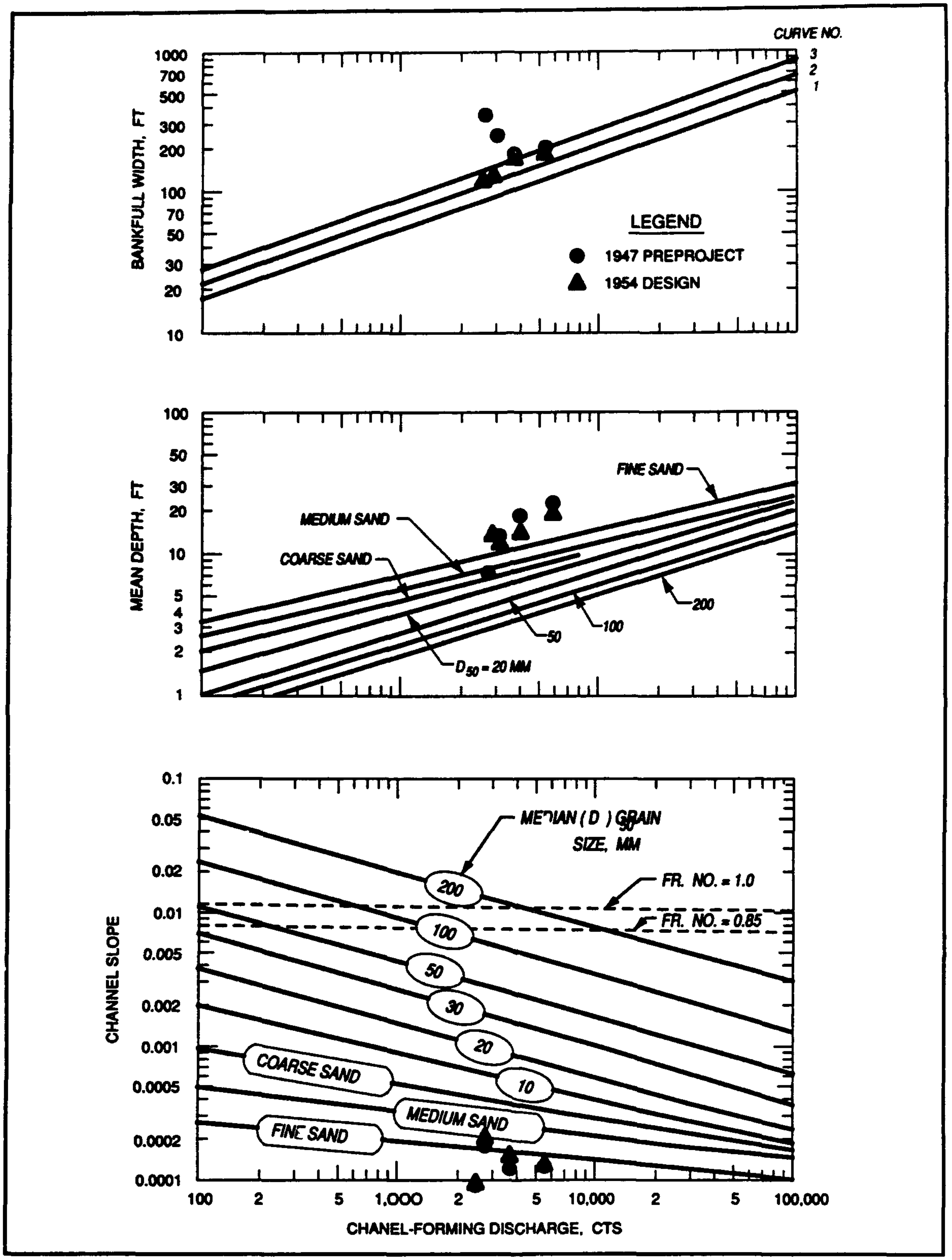

Figure 6. Hydraulic geometry, 1947-1954 channel dimensions

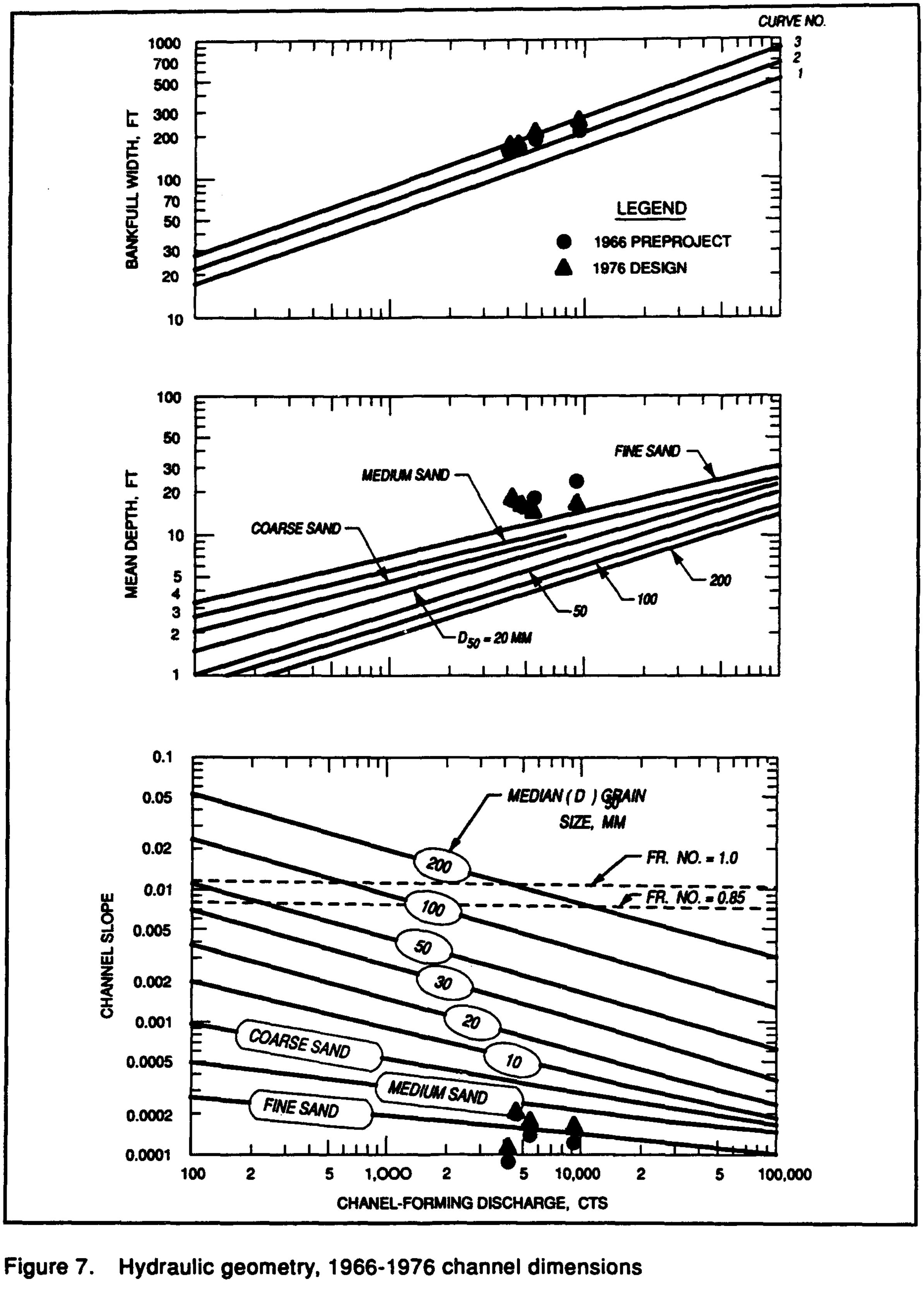

Figure 7. Hydraulic geometry, 1966-1976 channel dimensions

preproject and design conditions. The prototype was both wider and deeper than predicted using the charts. This inconsistency was attributed to the effect of heavily vegetated banks in the natural stream, which is unaccounted for in hydraulic geometry charts. Predicted slopes from the charts were scattered about the predicted slope line for fine sand. Due to the large scatter, the applicability of the slope chart could not be confirmed for Big and Colewa Creeks.

It was concluded that stable channel top widths could be obtained from the hydraulic geometry chart using a line midway between curves 2 and 3. Depth and slope would have to be calculated separately.

White, Paris, and Bettess Analytical Method

The White, Paris, and Bettess method can be applied to evaluate channel stability by using one of the "CORPS system"[1] programs H9121. Input requirements for this program are kinematic viscosity, specific weights of sediment and water, d_{35} of the bed, discharge, and sediment concentration. Sediment concentration can be calculated using the SAM program. The White, Paris, and Bettess program was developed using the Ackers-White sediment transport function; sediment concentration can be calculated using the Ackers-White d_{50} option in SAM (with $d_{50} = d_{35}$). Input hydraulic parameters for SAM can be calculated with a variety of methods including normal depth, or the HEC-2 backwater program. In this study, hydraulic parameters were determined for each reach, and sediment concentrations were calculated using the Ackers-White equation. Program H9121 was then used to calculate a stable channel geometry that would pass the sediment load from the upstream cross section. This procedure assumed that sediment equilibrium was attained in the upstream reach, an assumption that may not be valid in a creek with extensive cohesive outcrops. Results are shown in Table 3.

Using the White, Paris, and Bettess method, predicted slopes were significantly less than prototype slopes. This is attributed to the lack of consideration for bank roughness in the White, Paris, and Bettess method and to the assumption of minimum stream power, and hence minimum slope, inherent to the method. The difference between 1947 preproject calculated and prototype slopes is greater than between 1954 design calculated and prototype slopes because of the greater bank roughness in 1947. Calculated composite Manning's roughness coefficients from the White, Paris, and Bettess method are also considered to be low. The White, Paris, and Bettess method only considers grain and bed form roughness. Side slope predictions are inconsistent with design values determined

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1 Conversationally Oriented Real-Time Program System (CORPS) computer programs available from U.S. Army Engineer Waterways Experiment Station, ATTN: CEWES-IM-MI-C, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

Table 3
Stable Channel Geometry Using White, Paris, and Bettess Method[1]

Reach No.	Discharge cfs	Concentration ppm	Velocity fps	Base width, ft	Depth of flow, ft	Side slope, H/V	Channel slope	Manning's n
1947 Preproject Condition								
1	5300	31	2.5	81	18.6	1.8	0.000053	.024
2	3600	39	2.5	67	15.7	1.6	0.000064	.024
3	2800	6	2.0	57	17.0	1.5	0.000031	.022
1954 Design Condition								
1	5300	98	2.9	81	16.5	1.8	0.000091	.026
2	3600	202	3.2	65	13.2	1.6	0.000143	.026
3	2800	35	2.4	59	14.4	1.5	0.000065	.024
1966 Preproject Condition								
1	9100	122	3.2	106	19.6	2.0	0.000091	.027
2	5500	269	3.4	80	15.0	1.8	0.000151	.027
3	4600	57	2.7	75	16.6	1.7	0.000073	.025
1976 Design Condition								
1	9100	151	3.3	105	19.2	2.0	0.000101	.027
2	5500	248	3.4	80	15.1	1.8	0.000145	.027
3	4600	60	2.7	75	16.5	1.7	0.000074	.025

[1] Kinematic viscosity = 0.0000106 ft^2/sec.
Specific weight of sediment = 165 lbs/ft^3.
Specific weight of water = 62.4 lbs/ft^3.
d_{35} of bed material = 0.17 mm.

from soils investigations. Design side slopes ranged between 2.0 in reach 4, to 3.0 in the downstream reaches. It is concluded that use of the White, Paris, and Bettess method is severely restricted in Big and Colewa Creeks due to its low slope and roughness predictions.

Stability Analysis Using SAM

The stable-channel analytical method in SAM was used to evaluate existing channel stability. Using the Brownlie equation, sediment inflow concentration was calculated from geometry of the upstream supply reach. It was assumed that the upstream section carried its equilibrium sediment load. Due to the high percentage of cohesive material found in the bed of Big and Colewa Creeks, especially in the upstream reaches, this assumption may not be appropriate. Since the channel-forming discharge was always larger in the downstream reach than in the upstream reach, a discontinuity with respect to sediment inflow occurred. This was addressed by using both a low and high sediment inflow condition. The low case represented a condition where inflow from tributaries was essentially sediment free, and sediment inflow to SAM was based on the calculated volume of sediment inflow from the upstream reach. The high case represented a condition where sediment contributions from the tributaries had a concentration equal to that from the upstream reach. Input to the stable-channel analytical method in SAM was

a. Median grain diameter = 0.2 mm.

b. Gradation coefficient = 1.5.

c. Water temperature = 60 °F.

Hydraulic input was from Table 1. In this analysis it has been assumed that the typical cross section can be represented by a trapezoidal section and that all of the channel-forming discharge was contained in the channel. This assumption may not be appropriate for the 1947 natural channel where significant flooding occurred during the 1- to 2-year return period flood.

Stability curves for 1947 preproject conditions are shown in Figure 8. Sediment supply from reach 4 is small when compared with the transport capacity in reach 3. As expected, the natural channel plots in the zone where degradation is predicted. This indicates that this reach is unstable; and that the stream is either degrading and narrowing, or that a headcut is moving through the reach. This conclusion does not consider the possibility of a natural cohesive bed control. In reach 2, the 1947 natural channel condition plots in the degradation zone; however, it is closer to a stable condition than in reach 3. These curves are based on the assumption that equilibrium sediment transport is attainable in the upstream reach and that all of the channel-forming discharge is retained in the channel. In

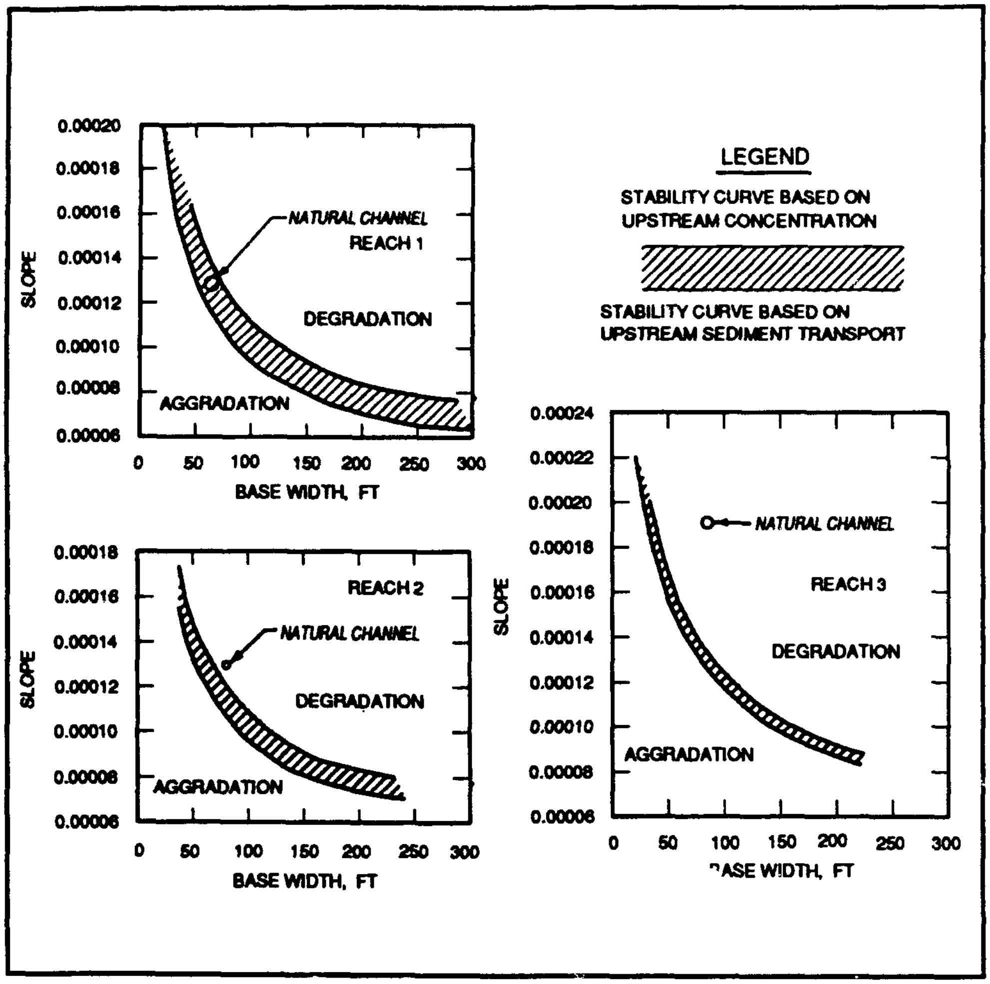

Figure 8. Stability curves for 1947 preproject conditions

reach 1, the 1947 natural channel plots within the zone of predicted channel stability. The conclusion of the stability analysis of 1947 preproject conditions is that the natural channel in reaches 2 and 3 has a tendency to degrade or that some type of natural stabilization was present in these reaches.

The effect of 1947 to 1954 project improvements on channel stability were evaluated using the SAM stable-channel analytical method; stability curves are plotted in Figure 9. Reach 4, the upstream supply reach, was channelized so that sediment transport potential was increased. However, cutoffs and channel improvements also improved the sediment transport potential in reach 3. The net result is that the design channel in reach 3 continued to plot in the degradation zone. The condition is similar to the preproject condition, and depending on the nature of natural stabilization such as cohesive bed control, in this reach, there may be no need for concern with respect to stability. However, channel stabilization techniques may be required. Low-water weirs were included in the channel improvement scheme and provided some measure of grade control. Sediment

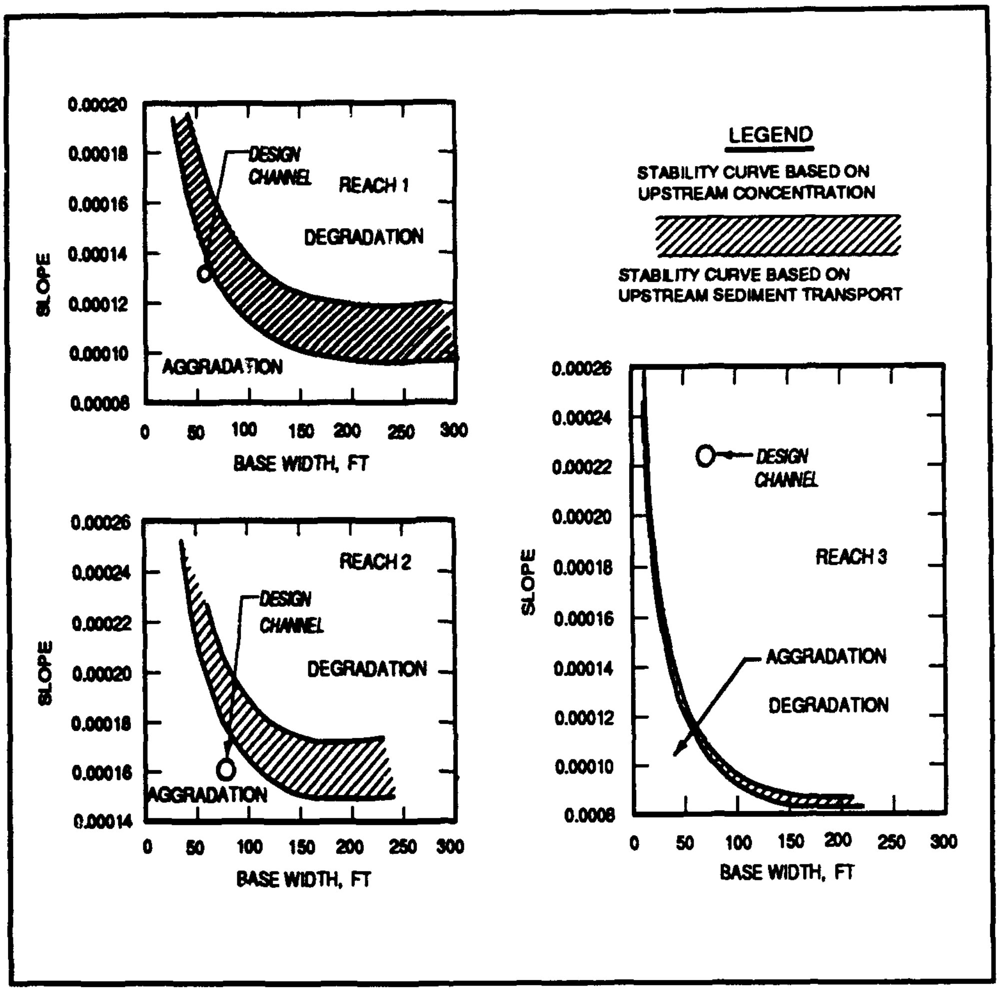

Figure 9. Stability curves for 1954 design conditions

transport potential was increased in reach 3 more than in reach 2 by the 1947 to 1954 improvements. As a result, the design channel plots in the aggradation zone. The stability analysis suggests that a wider channel, or more cutoffs would be appropriate. It is important to remember that this conclusion is based on the assumption of equilibrium transport in reach 3. The project design channel in reach 1 also plots slightly into the aggradation zone. It is close enough to the stability curve to be considered stable. The conclusion of the stability analysis of the 1947 to 1954 project design is that the channel will continue to have degradation potential in reach 3 and that the project will reverse its degradation trend in reach 2, with slight aggradation in both reaches 2 and 1. The severity of this aggradation trend will depend on natural stability features that may exist in reach 3.

In 1966, the channel was assumed to be essentially the same as it was in 1954; however, channel-forming discharge had increased. The effect of the increase in discharge on the stability curves is shown in Figure 10. Comparing Figures 9 and 10, it is apparent that the change in

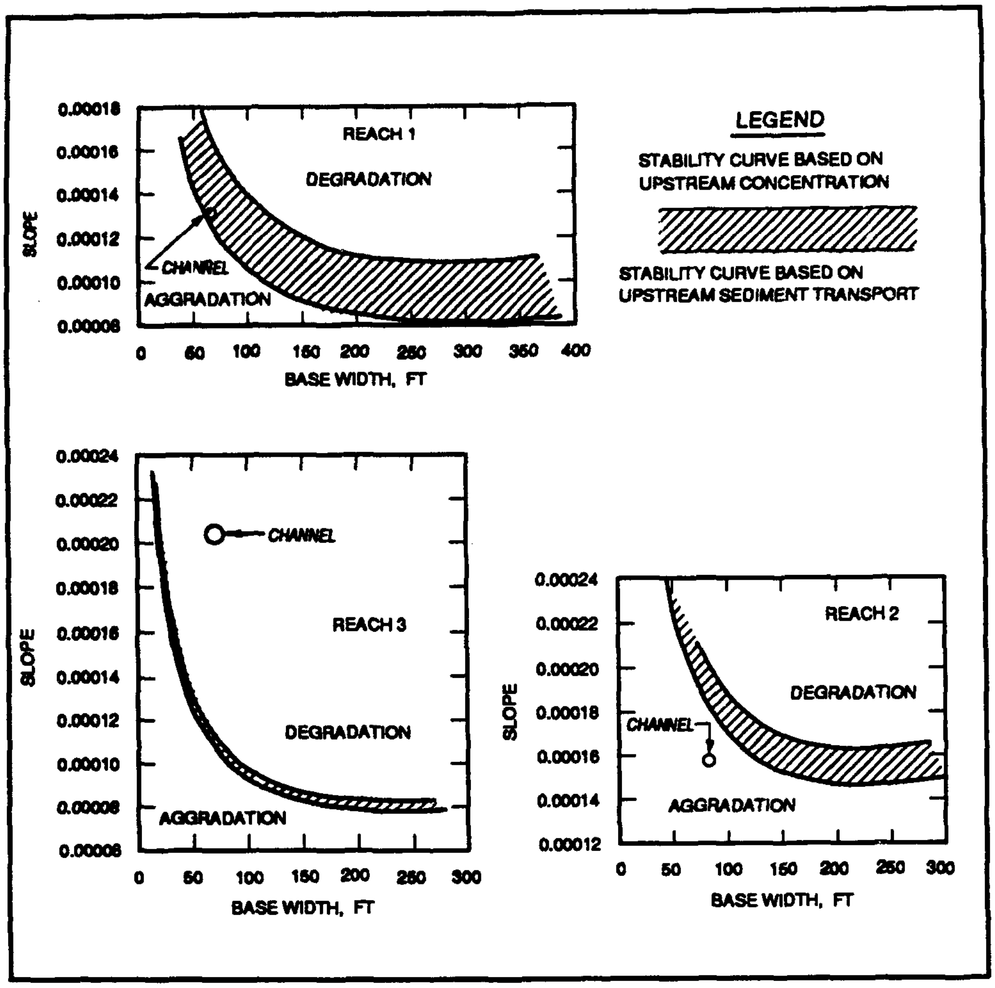

Figure 10. Stability curves for 1966 preproject conditions

channel-forming discharge did not significantly affect the relationship of the channel to the stability curves. It is probable, however, that the quantity of degradation and aggradation will increase with the increase in channel-forming discharge, so that whatever changes occur will be more significant.

Project improvements from 1966 to 1976 included channel widening and cutoffs in reaches 1 and 2. The relationship of the design channel to the stability curves is shown in Figure 11. These curves indicate a continuing potential for degradation in reach 3. The 1966 to 1976 design channel in reaches 1 and 2 results in a degradation trend in both reaches. This suggests a need for grade control.

The reliability of the stable channel analytical method cannot be confirmed in Big and Colewa Creeks from available data. The method predicted a degradation trend in reaches 1, 2, and 3, for post 1976 conditions. Prototype documentation describes aggradation in reach 3 but overall degradation in the system. The aggradation can be explained as a local

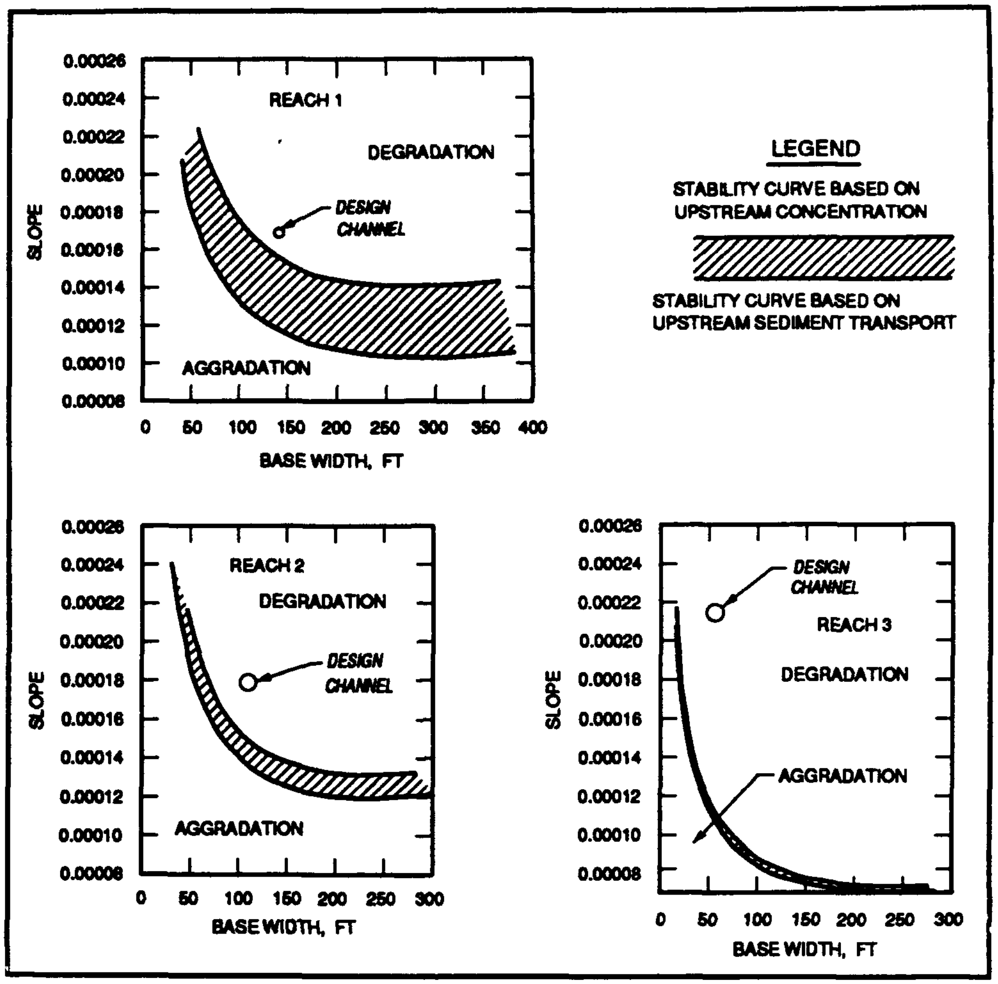

Figure 11. Stability curves for 1976 design conditions

problem due to tributary sediment inflow or from head cutting. The lack of degradation in reaches 1 and 2 can be explained by possible cohesive bed material or by the numerous low flow weirs that act as grade control. It is interesting that sediment does not fill behind the low-flow weirs to the crest elevation, indicating a sediment starved system. The stability analysis clearly does not account for natural or artificial grade control, and its application to streams with significant bed cohesion is questionable.

Application for Design — Critical Velocity

Channel dimensions from the 1966 recommended project were used to calculate velocities and depths. Average velocities, listed below, are less than 3 fps. According to criteria in EM 1110-2-1418, this should be a stable channel. These velocities are similar to those determined for the 1954

design channel and the 1966 preproject channel. Based on this assessment alone, it could be concluded that the channel is stable. This project was completed only in reaches 1 and 2, so it is not possible to determine conclusively whether or not this conclusion would have been correct. It is known that the channel has an overall tendency for degradation, even with the system of low-flow weirs.

Reach No.	Discharge cfs	Velocity fps	Base Width ft	Side Slopes H/V	Depth ft
1	9100	2.6	140	3	17.5
2	5500	2.5	110	3	14.0
3	4600	2.4	110	3	12.5
4	4200	2.4	110	3	11.5

Application for Design — Hydraulic Geometry

It was determined from the channel stability assessment that prototype widths fell between curves 2 and 3 on the hydraulic geometry width prediction chart. In this case, it appears appropriate to use the width prediction charts to determine a design channel top width and to assign or calculate slopes and depths. For the example using hydraulic geometry techniques, design top widths were taken from the hydraulic geometry charts. Slopes proposed for the 1966 to 1976 project were chosen. Side slopes of 3.0 had been determined separately from soil investigations. A composite Manning's roughness coefficient of 0.035 was chosen based on the 1966 design documents. These design parameters and calculated hydraulic parameters are listed below.

Reach No.	Discharge cfs	Top Width ft	Slope	Velocity fps	Base Width ft	Depth ft
1	9100	220	.00017	3.1	115	17.7
2	5500	170	.00018	2.8	75	15.8
3	4600	160	.00022	2.9	80	13.3
4	4200	150	.00011	2.3	23	21.1

In this example it is apparent that the hydraulic geometry charts could reliably provide only a single channel dimension, and that other techniques were required to complete the design. Reliable use of the width prediction chart required data from the existing channel to establish a reliable relationship between width and channel forming discharge.

Application for Design — White, Paris, and Bettess

Big and Colewa Creeks have a unique longitudinal slope condition in that channel slopes tend to increase in a downstream direction. This is indicative of base level control in the river system or in decreasing channel size in the downstream direction. These conditions are not accounted for in the analytical methods described herein, which are based on the principle of equilibrium sediment transport. For purposes of the design example, it will be assumed that the design objective is a channel with a sand bed, in equilibrium. The supply reach is reach 4. The following design channel is chosen for this reach based on its ability to contain the design flood:

a. Base width = 110 ft

b. Side slope = 3H/1V

c. Bank roughness coefficient = 0.050

d. Bed slope = 0.00009

The Ackers-White equation was used to calculate equilibrium sediment transport for this cross section. The CORPS program H9121 was then used to calculate a stable channel geometry for the three downstream reaches. Two sediment inflow conditions were considered: a high sediment inflow concentration assuming that tributaries contribute the same concentration as reach 4, and a low sediment inflow concentration, assuming that the tributaries contribute no sediment. Calculated stable channel geometries are listed below:

Reach No.	Discharge cfs	Conc ppm	Slope	Base Width ft	Side Slope H/V	Depth ft	Velocity fps
1	9100	46	0.000056	107	2.0	21.8	2.8
2	5500	46	0.000063	82	1.8	18.1	2.7
3	4600	46	0.000066	75	1.7	17.0	2.6
1	9100	21	0.000039	106	2.0	23.6	2.5
2	5500	35	0.000055	83	1.8	18.6	2.6
3	4600	42	0.000063	75	1.7	17.1	2.6

These designs call for significant grade control in order to achieve the prescribed bed slope. Base widths and channel side slopes were less than those constructed in the prototype; depths were higher.

Application for Design Using SAM

SAM was used to calculate stability curves for reaches 1, 2, and 3. Two curves are plotted for each reach: one assuming a constant volume of sediment supplied from the supply reach, and one assuming a constant concentration. These curves are shown in Figure 12. Stable channel geometries can be chosen from these curves for each reach. The choice may be affected by depth constraints or by width constraints. Channel geometry can be calculated using SAM by specifying a design slope in the program. Two possible channel designs are presented in the following tabulation:

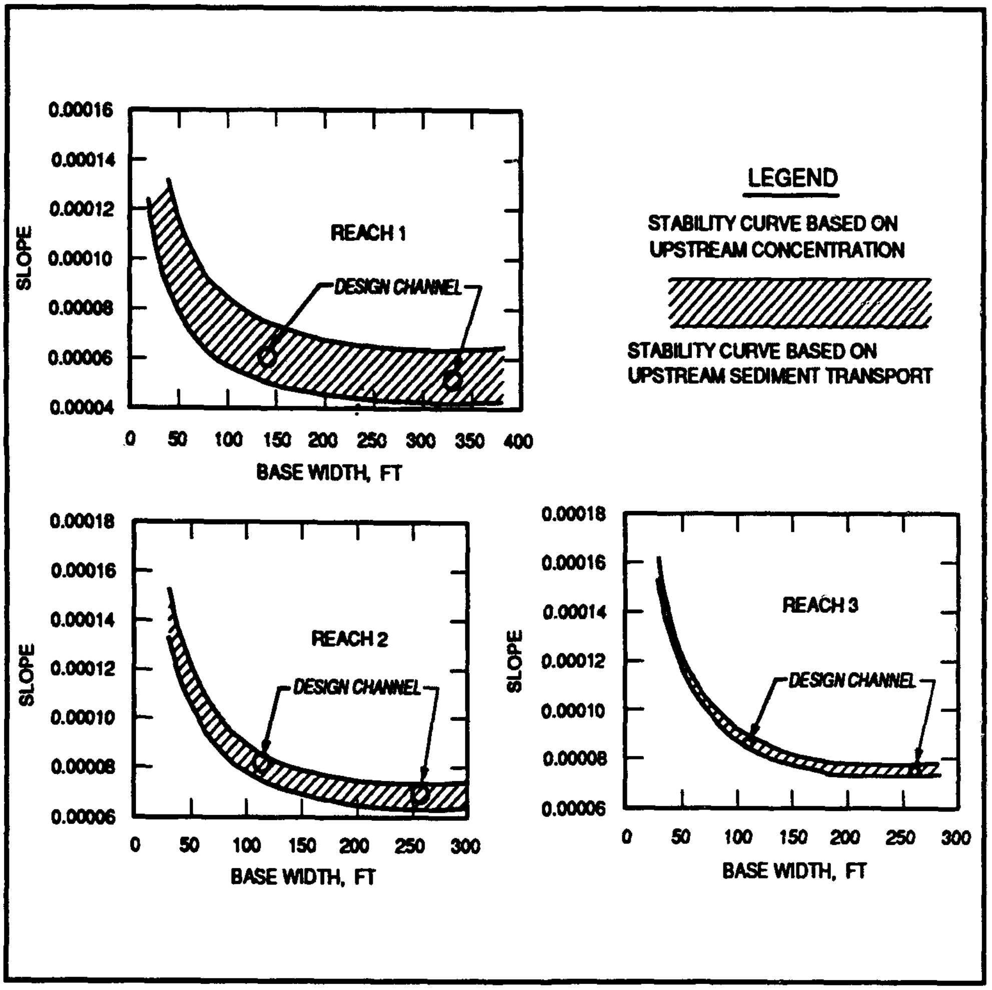

Figure 12. Stability curves for project design

Reach	Bed Slope	Base Width ft	Side Slope H/V	Depth ft	Velocity fps	Conc ppm
1	0.000053	330	3	13	1.9	23
2	0.000069	260	3	10	1.9	28
3	0.000075	240	3	9	1.9	30
4	0.000090	110	3	14	1.9	32
1	0.000062	140	3	22	2.0	23
2	0.000079	120	3	17	2.0	28
3	0.000087	110	3	16	1.9	30
4	0.000090	110	3	14	1.9	32

These designs also call for significant grade control, although not as much as was prescribed with the White, Paris, and Bettess method.

Conclusions

The stable channel techniques described in EM 1110-2-1418 have limited applicability for streams with significant bed control due to cohesion. The analytical methods of White, Paris, and Bettess, and SAM are especially affected by disequilibrium sediment transport. These analytical methods can only be used to determine probable trends. Both methods indicated degradation in the channel after flood control improvements. The prototype channel has displayed a tendency for degradation, even though grade control and cohesive bed materials tend to limit the severity of degradation.

The critical velocity approach appeared to be adequate for determining channel stability in Big and Colewa Creeks. It is somewhat discomforting, however, that the degradation trend observed in the prototype was not identified.

The White, Paris, and Bettess method and the hydraulic geometry charts did not appear to adequately account for the effect of bank roughness.

3 Case Study, Puerco River

Introduction

The Puerco River is a high-energy, ephemeral stream that flows through Gallup, New Mexico. The sand-bed stream is normally dry but is subject to flash floods, with very high sediment loads. Suspended sediment concentrations up to 500,000 ppm have been recorded. The Puerco River has a drainage area of about 550 square miles and a slope varying between 0.0020 and 0.0030 through an 8-mile study reach. The study reach extends from a bedrock grade control, locally called the rock knoll, located about 3 miles downstream from Gallup, for about 9 miles to the confluence of the north and south forks of the Puerco River (Figure 13). Through Gallup, the floodplain has been extensively developed or closed off by road and rail embankments and levees. The AT&SF railroad embankment runs along the south bank of the river, and for most of the study reach the river is bounded on the north by Interstate Highway 40 (I-40). The channel bottom varies in width between 50 and 150 ft and is essentially barren of vegetation. The channel has a fine sand bed with a median grain diameter of about 0.17 mm. The banks are either actively eroding or sparsely covered with desert riparian growth. Bank materials include stratified silt and fine sand and occasional layers of cemented sand or gravel.

At mile 1.3, the rock knoll and the AT&SF railroad embankment constrict the Puerco River channel to a width of about 75 ft (Figure 14). The channel upstream is between 100 and 150 ft wide and has entrenched 4 to 6 ft into a wide floodplain. The channel slope between the rock knoll and Gallup varies between 0.0022 and 0.0025. Through Gallup, between miles 3.5 and 4.5, a soft-bottom channel with concrete side slopes was constructed by the New Mexico State Highway Department during the late 1970's (Figure 15). The new channel varied in width between 110 and 160 ft and was originally 10 ft deep. The design slope was 0.00179 at the downstream end and 0.00213 at the upstream end. The channel has aggraded between 3 and 5 ft since its construction. The existing slope through this reach is between 0.0019 and 0.0021. The Gamerco Spur railroad bridge crosses the channel in this reach, and the aggradation has severely restricted the clearance under the bridge (Figure 16). Between miles 4.5 and 6.5, the channel bottom width varies between 50 and 125 ft, and the

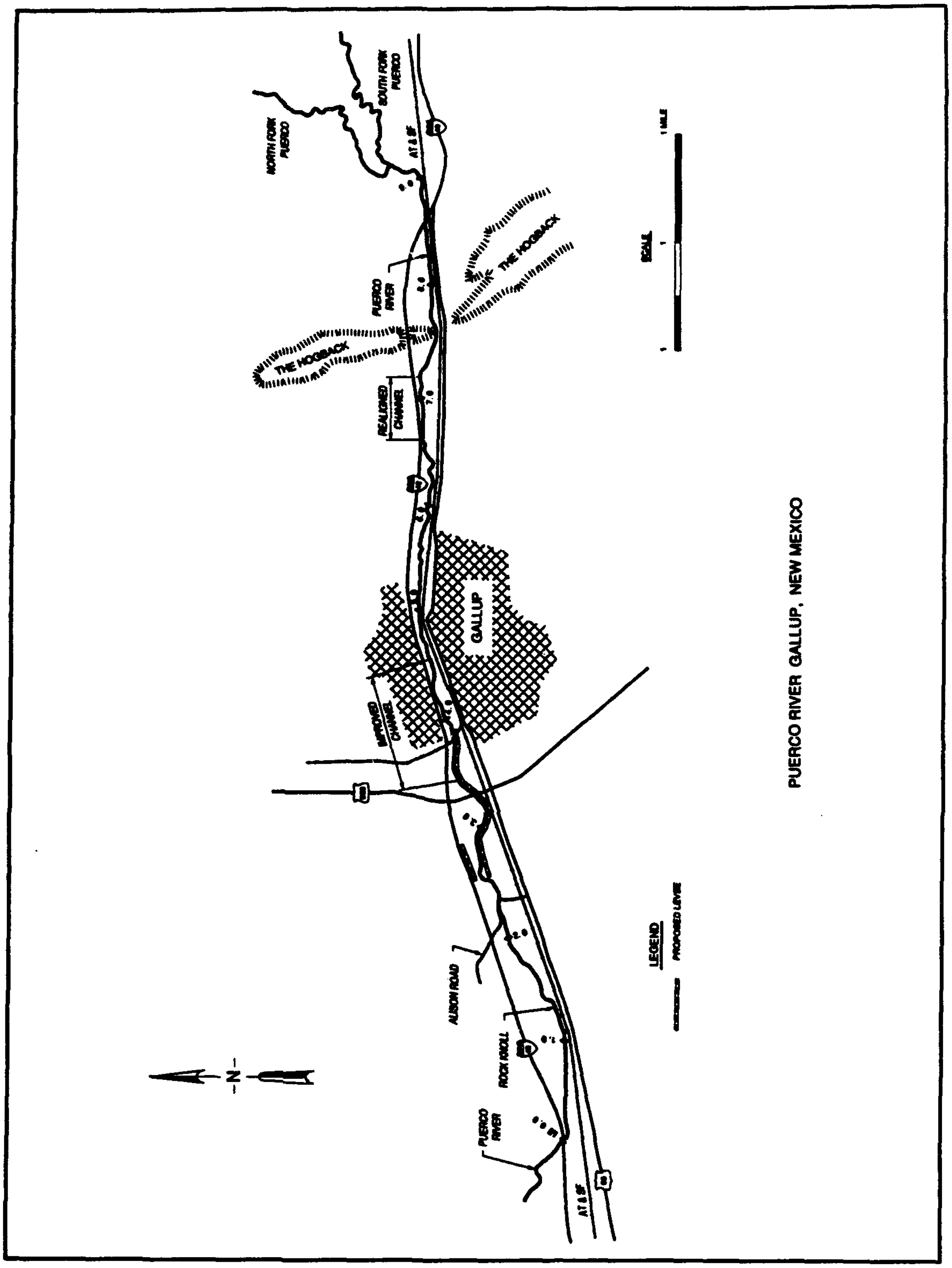

Figure 13. Puerco River study reach

Figure 14. Upstream view of the rock knoll and the railroad embankment - river mile 1.3

Figure 15. Upstream view of the improved channel - river mile 4.0

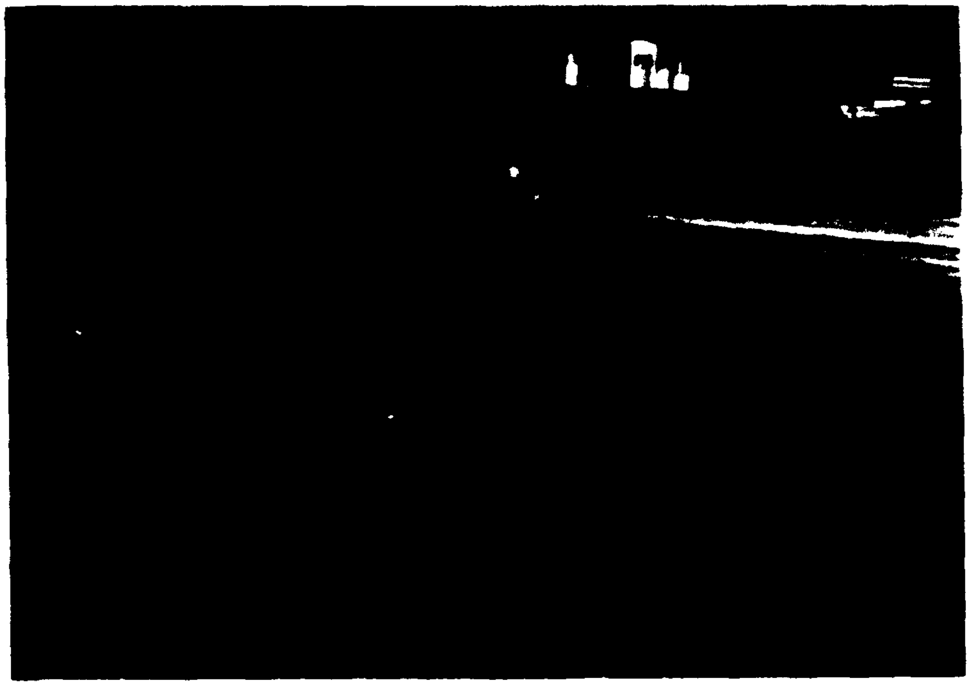

Figure 16. Gamerco Spur railroad bridge - river mile 3.8

Figure 17. Upstream view of improved channel - river mile 5.5

channel is deeply entrenched, from 10 ft downstream to 30 ft upstream. When I-40 was constructed, the New Mexico State Highway Department realigned the Puerco River between miles 6.5 and 7.5. A river reach of about 9,000 ft was shortened to about 5,000 ft. Three sheet-pile drop structures were constructed in this reach to stabilize the invert and Kellner Jacks were installed to stabilize the banks (Figure 18). The Hogback, a rock ridge at about mile 8.0 that dips under the Puerco River channel, provides grade control and constricts the channel to a width of about 70 ft against the AT&SF embankment. In general, the river appears to be degrading in the upper reaches, and aggrading in the lower reaches. A channel profile is shown in Figure 19.

The Corps of Engineers designed a flood-control project for Gallup that included improvement and extension of existing levees and removal of a portion of the rock knoll that constricts flow and provides undesirable grade control. The design discharge was 20,000 cfs, which has a 100-year frequency.

The primary stability problem associated with this project is the tendency for aggradation in the project reach. This is especially critical due to the restriction under the Gamerco Spur bridge. It is also desirable to prevent aggradation between the rock knoll and the project.

The river was divided into 10 reaches for the stability analysis. Typical hydraulic parameters were determined for each reach and are listed in Table 4. Reach 1 extends from the confluence of the north and south forks of the Puerco River to the grade control at the Hogback. Reach 1 was established as the supply reach for the stability analysis (Figure 20). Channel geometry in this reach was defined by 5-ft contour topographic maps. Reach 2 extends from the Hogback to the upstream end of channel realigned by the highway department. Two-foot contour mapping taken in 1988 was used to determine channel geometry for reach 2 and all reaches downstream. Reach 3 is the realigned reach and contains three drop structures; the average slope in this reach was adjusted to account for the drops. Reach 4 extends from the downstream end of the realigned channel to a grade break in the natural channel located about 3,000 ft from the upstream end of the downtown channel improvements. Reach 5 is the 3,000-ft length of natural channel upstream from the downtown channel improvement. Reaches 6 and 7 are in the downtown improved channel and have significantly reduced slopes relative to the upstream reaches. Reach 8 extends from the downstream end of the improved channel to Highway 666. Reach 9 extends from Highway 666 to Alison Road and is somewhat steeper than reaches upstream and downstream. Reach 10 is the downstream-most reach extending from Alison Road to the rock knoll. This reach also has a mild slope controlled by the rock outcrop.

Table 4
Average Hydraulic Parameters

Reach No.	River Mile	Base Width, ft	Slope	Side Slope, H/V	n_{bank}
	9.0				
1		75	0.0028	1.5	0.050
	7.8				
2		90	0.0026	1.5	0.050
	7.0				
3		80	0.0027	1.5	0.050
	6.4				
4		87	0.0025	1.5	0.050
	4.8				
5		77	0.00225	1.5	0.050
	4.3				
6		125	0.0021	2.0	0.030
	3.8				
7		160	0.0019	2.0	0.030
	3.4				
8		140	0.0019	1.5	0.050
	3.0				
9		150	0.0025	1.5	0.050
	2.2				
10		100	0.0022	1.5	0.050
	1.2				

Channel Stability

It is difficult to determine the stability of the existing (1988) channel. Comparing channel surveys is difficult due to the different accuracies in historical surveys. It is conclusive that the improved channel through the city of Gallup aggraded significantly since its construction in the mid 1970's. Comparisons of 1984 and 1988 topographic surveys indicated degradation in reaches 3, 4, and 10, with other reaches remaining essentially stable. Several sediment studies have concluded that the improved channel will degrade during a major flood, ensuring channel capacity.

The definition of channel stability is important in a river like the Puerco. In terms of bank stability, the river is clearly unstable, with severe bank erosion apparent throughout the study reach. In this analysis, stability is defined as the absence of a tendency for aggradation or degradation. Even this definition is difficult to establish because the bed of the river may react differently, depending on the magnitude of the discharge. In this analysis, the 10-year peak discharge of 8,600 cfs was chosen as the characteristic discharge for determining stability.

It is difficult to define a typical cross section for most of the reaches in this study because channel width varies extensively. Some cross sections were not easily reduced to a simple trapezoidal shape. Base widths for the typical trapezoidal cross section should be representative of the movable-bed width of the channel. Sometimes this included a bench or bar in the cross section. Prototype observations were required to make these distinctions. Typical cross sections for each reach were determined by averaging base widths and side slopes from each section in the reach. Average side slopes in the natural sections were found to be close to 1.5 in all reaches,

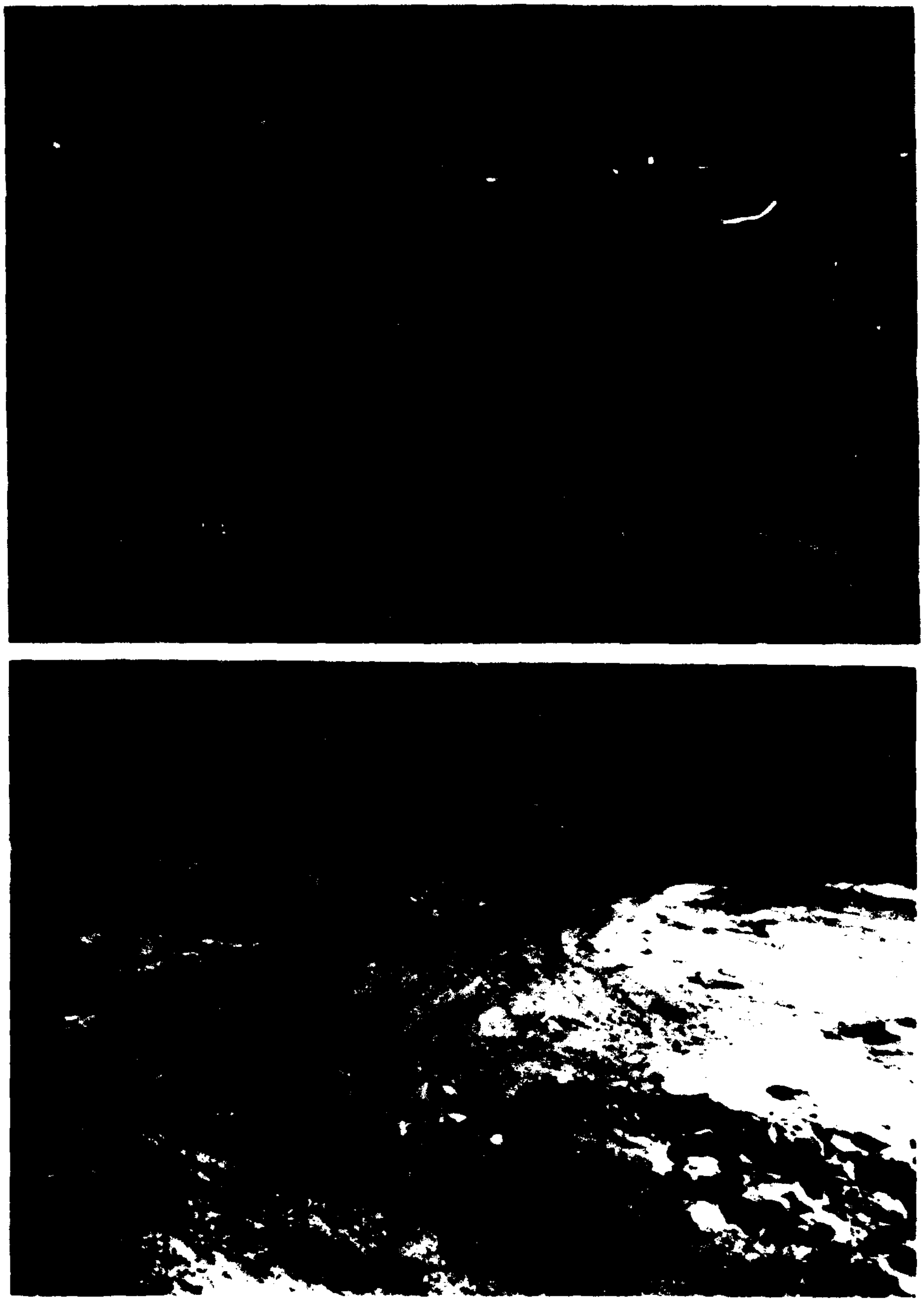

Figure 18. Realigned channel reach showing sheet-pile drop structure - river mile 6.6

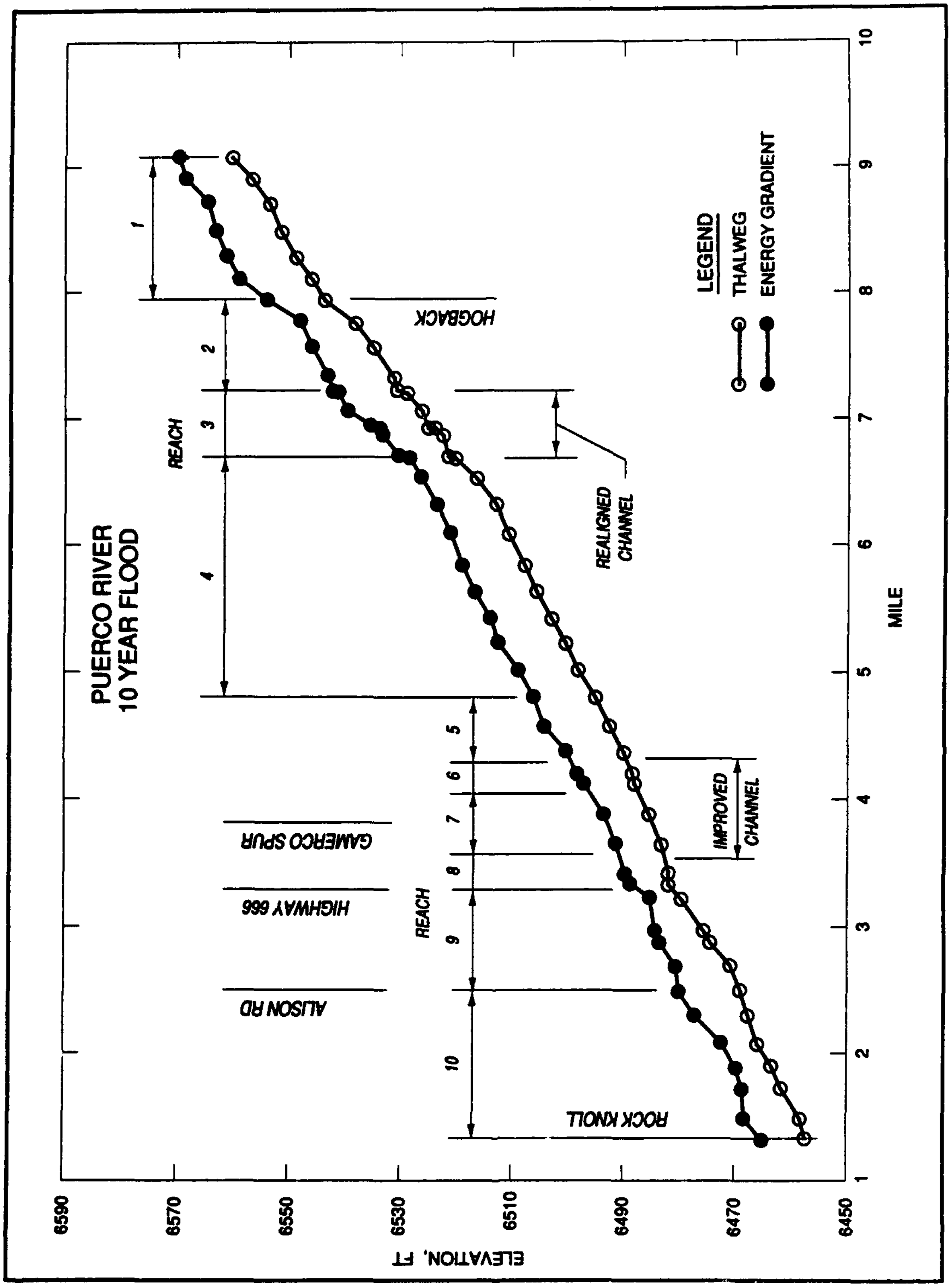

Figure 19. Puerco River thalweg and 10-year peak energy slope

and this value was assigned to all natural reaches. Average hydraulic parameters for each reach were determined from an HEC-2 water-surface profile calculation for the 10-year peak flood of 8,600 cfs.

Critical Velocity and Shear Stress

The critical velocity and critical shear stress methods were found to be inapplicable for the Puerco River. Average channel velocities for the 10-year peak ranged between 6 and 11 fps. These were significantly in excess of recommended critical velocities for fine sand beds. Dimensionless shear parameters vary between 7 and 23, also significantly in excess of recommended values.

Hydraulic Geometry

Calculated top widths for the 10-year peak ranged between 100 and 220 ft. This range of values falls between curves 1 and 2 on the hydraulic geometry width prediction chart. Banks contain stratified silt and fine sands and occasional layers of cemented sand or gravel, that would be expected to align between curves 2 and 3. Calculated 10-year depths ranged between 6 and 10 ft. These depths are less than the 14-ft value predicted using the depth prediction chart. The channel slope predicted from the slope hydraulic geometry chart is off by more than an order of magnitude. It was concluded that the hydraulic geometry method is not applicable to high-energy, ephemeral sand-bed streams.

White, Paris, and Bettess Method

Sediment inflow concentration was calculated using the Ackers-White sediment transport function. Hydraulic input parameters were based on the average cross section for the supply reach (reach 1) determined from topographic maps and backwater calculations. Using a d_{35} of 0.13 mm in the Ackers-White equation resulted in sediment concentrations in excess of 100 percent. Therefore, a sediment size of 0.17 mm, which represented the d_{50} size, was used in the sediment transport calculations. Predicted stable channel geometry for the 10-year peak is listed below:

a. Base width = 67 ft.

b. Velocity = 12 fps.

c. Depth of flow = 8.6 ft.

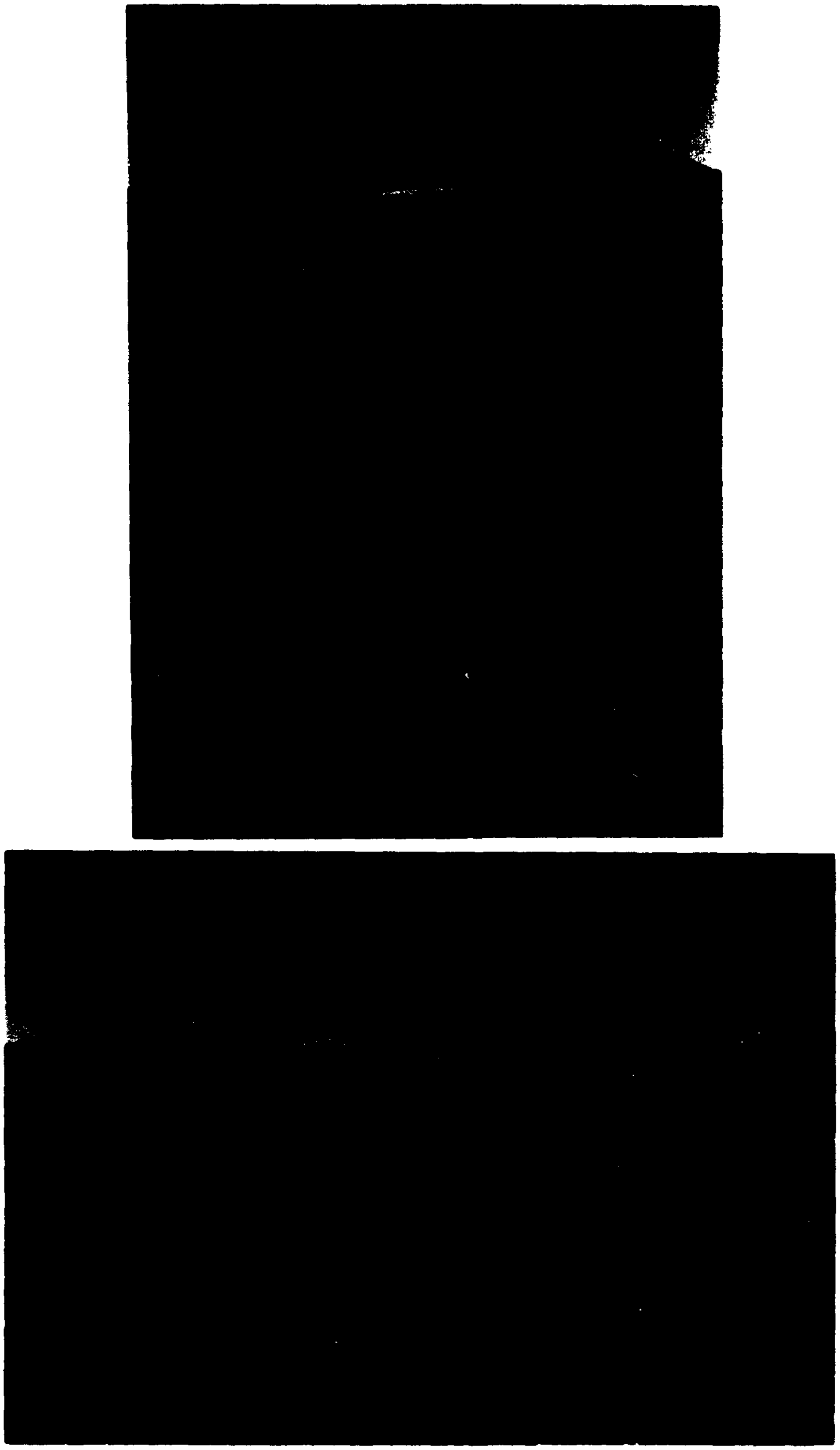

Figure 20. Puerco River upstream from Hogback - supply reach

d. Side slope = 2H/1V.

e. Slope = 0.00455.

f. Manning's n = 0.030.

g. Froude Number = 0.79.

The base width of the predicted channel is representative of narrow cross sections in the study reach. Predicted slope is steeper than any in the existing reaches of the river. This is probably due to the high calculated roughness. This method predicts a roughness coefficient of 0.030 compared with composite roughness coefficients between 0.025 and 0.017 using SAM. Froude Numbers in the prototype are closer to 1.0. The White, Paris, and Bettess method does not appear to be appropriate for this application.

Stable Channel Analysis Using SAM

Sediment inflow was calculated for reach 1 based on the principle of equilibrium transport. The roughness of the banks was considered to be the most uncertain of the input variables. Bank roughness must include all factors contributing to the roughness coefficient, other than bed form and grain roughness. These factors include the effects of channel irregularity, variations in cross-sectional shape, obstructions in the channel, and sinuosity. There is considerable variation in cross-sectional shape in the Puerco River, and the banks are very irregular. Bank roughness coefficients of 0.050 and 0.060 were assigned to the supply reach. The lower value was used to produce a "high sediment inflow" stability curve, and the higher roughness coefficient was used to produce a "low sediment inflow" stability curve. The two stability curves delineated a range to evaluate stability of the existing channel in the other reaches. Stability curves were developed for unimproved reaches with a side slope value of 1.5 and a bank roughness coefficient of 0.050. Stability curves were also developed for improved reaches with a side slope of 2.0 and a bank roughness of 0.030. Existing (1988) channel dimensions for the nine downstream reaches were compared with the calculated stability curves in Figures 21 and 22.

The stability curves can be used to evaluate the stability of the existing channel. Input parameters to SAM may have to be adjusted so that the stability curves provide an accurate representation of the existing channel conditions. In this case, the bank roughness in the supply reach was used as an adjustment parameter. Average base width and slope fell between the high and low sediment inflow stability curves in reaches 2, 3, 4, and 9 (Figure 21). These reaches were considered relatively stable. Reaches 5, 8, and 10 (Figure 21) fell below the stability curves in the aggradation zone. Reaches 6 and 7, in the improved portion of the river, fell below the

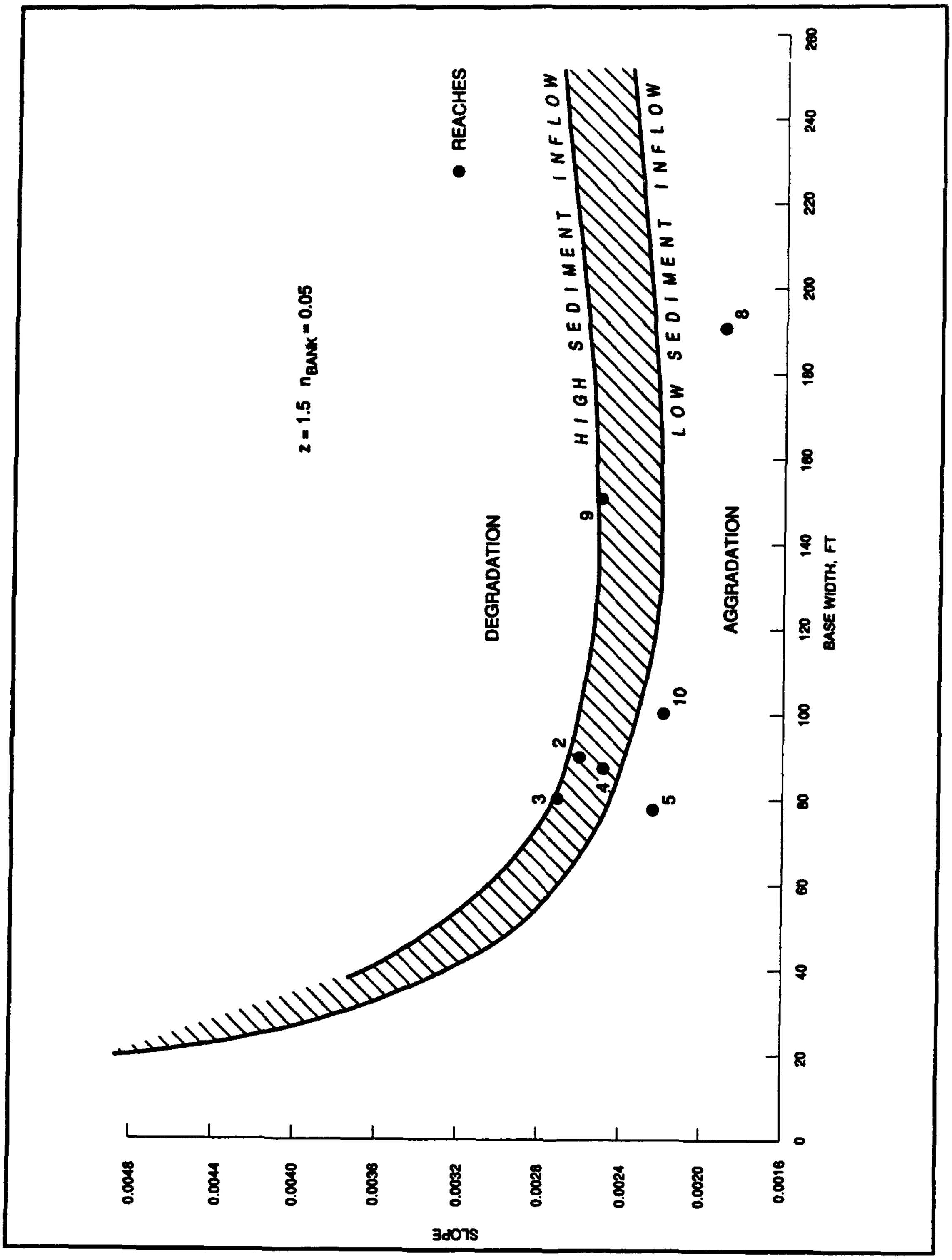

Figure 21. Stability curves for unimproved reaches, 10-year peak

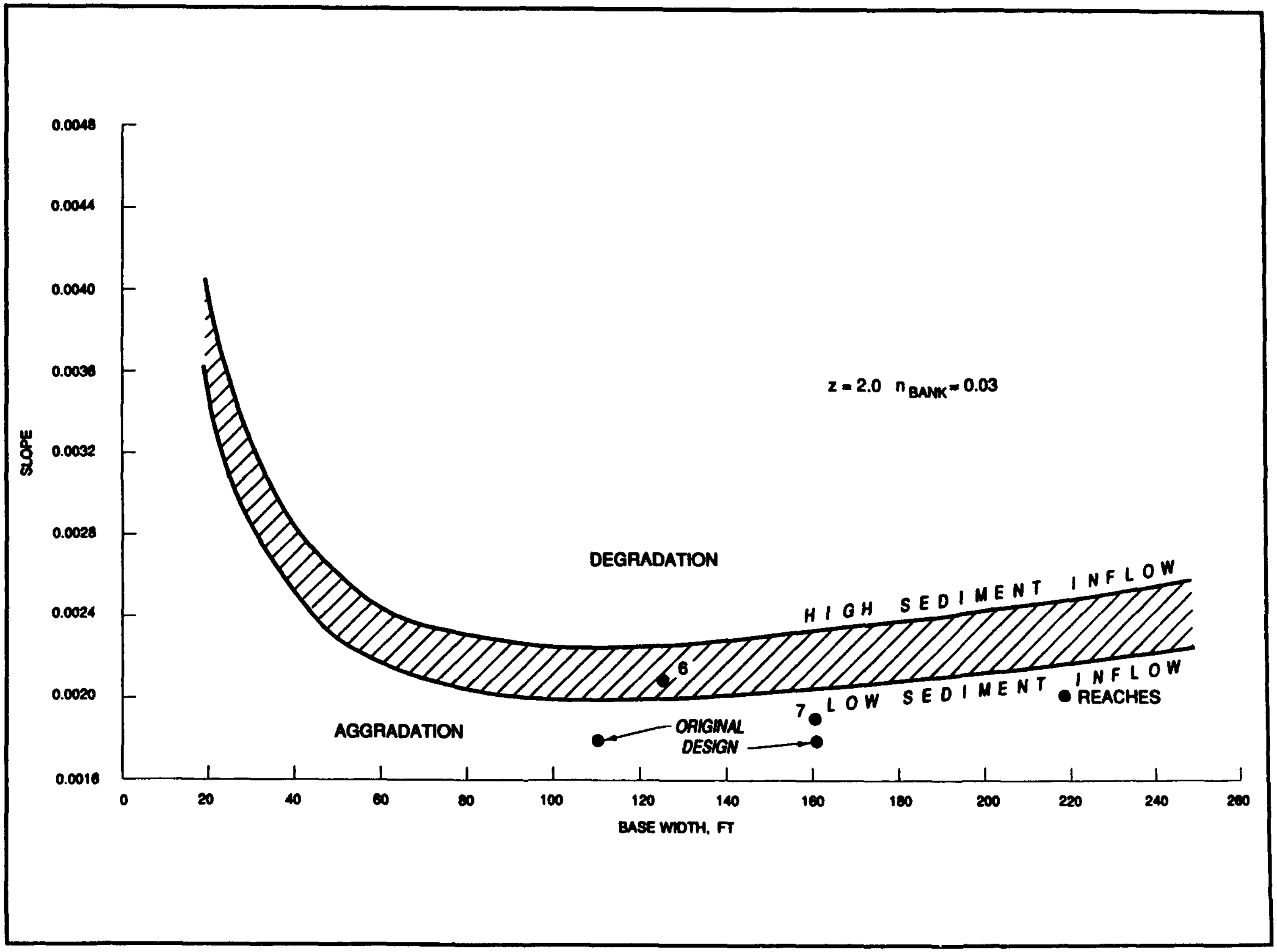

Figure 22. Stability curves for existing improved reaches, 10-year peak discharge

lower end of the stability range, indicating a slight aggradation trend (Figure 22). Slopes in this reach were based on the energy slope from backwater calculations and were slightly higher than bed slopes. The original design channel in the improved reach had a slope of about 0.0017. This slope was well within the aggradation zone as shown in Figure 22.

The preceding analysis was based on the 10-year peak as the channel-forming discharge. Stability curves for the 2-year flood were also developed for the improved channel reaches (Figure 23). These curves showed that the improved channel would have a severe aggradation tendency. Since the channel appeared to be relatively stable in this reach following initial aggradation after construction, it was concluded that the 10-year peak would be the more appropriate discharge to use for the channel-forming discharge.

Design Application

Based on the results of the stability assessment of methods using the existing channel, it was concluded that the stable channel technique in SAM was the only applicable technique for the Puerco River. The proposed project includes improving levees in reaches 6 and 7, and in lowering the elevation of the rock knoll to increase slope in the lower reaches.

Stability curves were plotted for the 2-, 10-, and 100-year peak discharges for the design channel. The high sediment inflow case was assumed; side slopes of 2H/1V and a bank roughness coefficient of 0.030 were assigned. The proposed design slope ranged between 0.00169 at the downstream end of the project to 0.00213 at the upstream end. The milder slope extended through the Gamerco spur bridge in order to obtain maximum clearance. Design channel geometry for the downstream and upstream end of the improved channel was plotted with the stability curves in Figure 24. Assuming that the 10-year peak is the channel-forming discharge, the stability curves indicate that reach 6 would have a slight aggradation tendency, but that reach 7 would have a more severe aggradation tendency. Comparing the cross sections with stability curves for the 2- and 100-year peaks, it can be seen that, for the 2-year peak, there would be a much greater tendency for aggradation, but for the 100-year peak, the upstream reach would have a slight degradation tendency, and the downstream cross section would have a decreased tendency for aggradation. This analysis does not quantify the severity of the aggradation problem. This quantification can be accomplished with more detailed hydraulic methods that employ the sediment continuity equation, including a sediment budget analysis or an HEC-6 numerical model.

The stability curves can be used to determine an appropriate design slope that would allow for general stability in terms of aggradation and degradation. Stability curves of the improved channel (Figure 22) indicate that a slope of about 0.0022 would provide a stable channel for

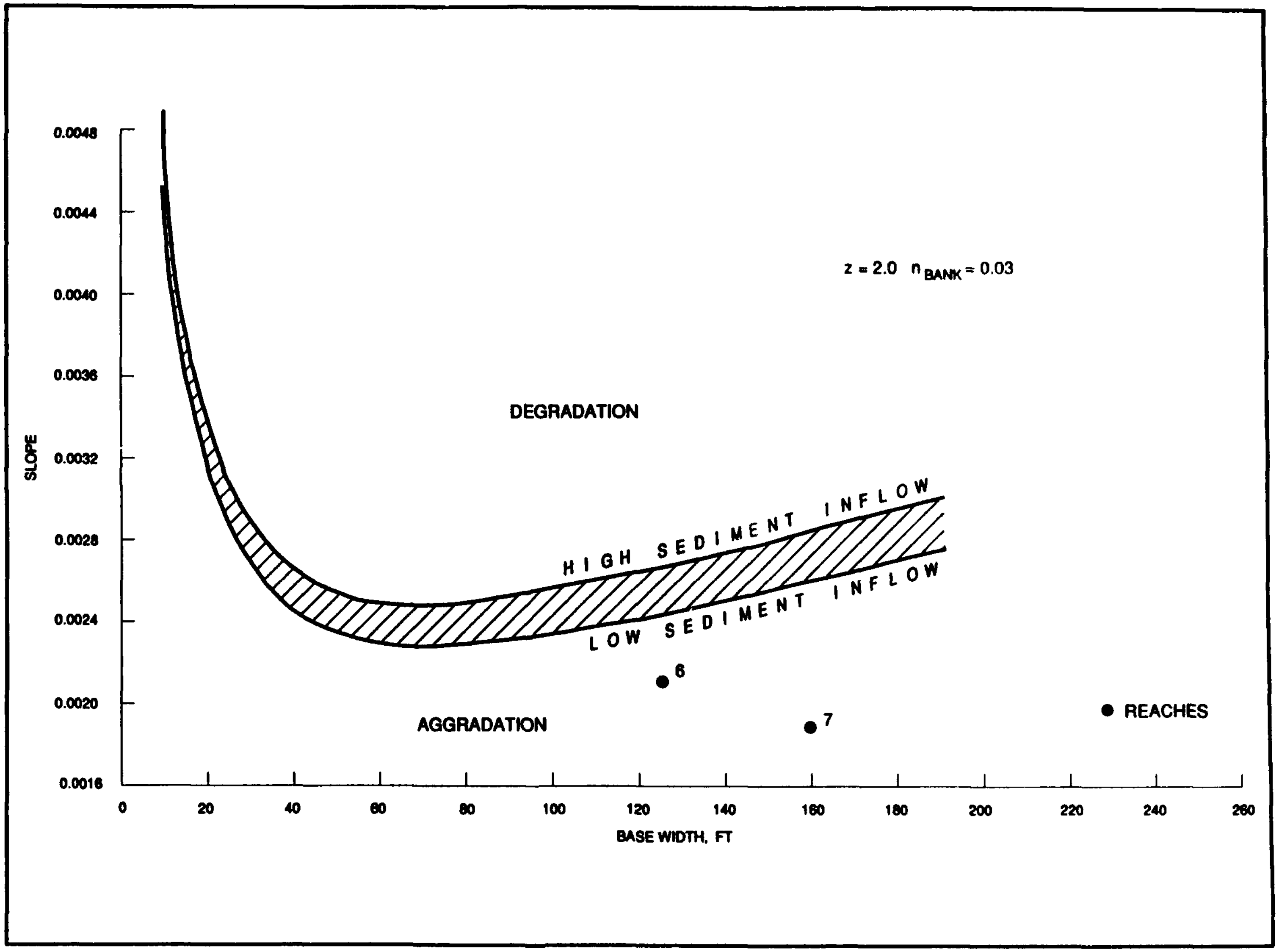

Figure 23. Stability curves for existing improved reaches, 2-year peak discharge

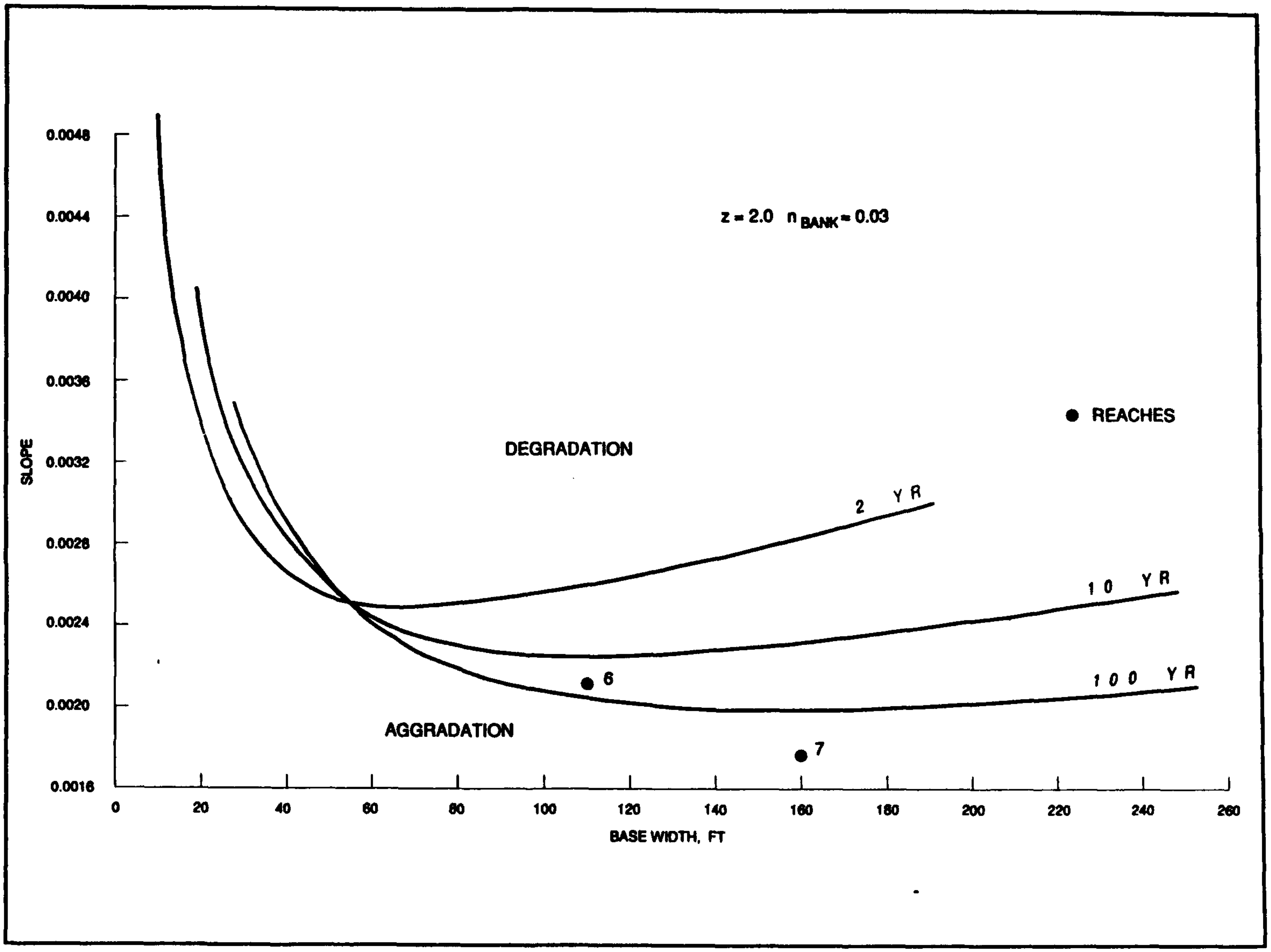

Figure 24. Stability curves for design channel - high sediment inflow

widths ranging between about 80 and 170 ft. For the unimproved reaches downstream, stability curves (Figure 21) indicate that a slope of about 0.0024 would provide a stable channel for widths ranging between about 110 and 200 ft. The distance between the rock knoll and the downstream end of the improved channel is about 12,000 ft, and the length of the improved channel reach is about 8,600 ft. After assigning the proposed slopes to these distances, a total drop in elevation of 47.7 ft was calculated. The existing drop between the upstream end of the improved channel and the rock knoll is about 40 ft. An 8-ft lowering of the rock knoll, accompanied by channel widening, would provide a stable channel between the rock knoll and the upstream end of the improved channel. Between the stable channel limits, channel widths would be determined based on right-of-way and hydraulic considerations.

Conclusions

The stable channel analytical method in SAM is applicable to high-energy, ephemeral streams like the Puerco River. Definition of the typical cross sections is critical to the analysis. Topographic mapping is not sufficient. Careful assessment of the movable bed width must be made, based on accurate survey data and observation of the prototype. Slope determination must account for the effects of drop structures and backwater due to constrictions or other obstructions. A backwater program such as HEC-2 can be used to determine energy slope. In streams such as the Puerco River, it is necessary to adjust the sediment inflow by adjusting hydraulic parameters for the supply reach until existing downstream reaches display reasonable responses. In this case study, it was the bank roughness in the supply reach that had a high degree of uncertainty and was used as an adjustment factor.

Appendix A
Stable Channel Analytical Method

Introduction

The Stable Channel Analytical method is a product of the Flood Control Channels Research Program (FCCRP). The focus of the FCCRP is to develop systematic methods for design of stable channels in small sand-bed streams. These methods are needed in the planning and preliminary design stage of local flood-control projects. In the detailed design phase of flood-control projects, other techniques, including numerical and physical modeling, are currently used. However, such detailed methods are often not required for evaluating the feasibility of a flood-control channel project in general or for comparing the performance of alternative designs.

To fill the gap between detailed numerical sediment models and the regime and threshold theories, an analytical approach has been developed for the design of stable channels. This analytical approach determines dependent design variables of width, slope, and depth from the independent variables of discharge, sediment inflow, and bed material composition. It involves the solution of flow resistance and sediment transport equations, leaving one dependent variable optional. Minimum stream power is used as a third equation for an optional fixed solution. This method is based on a typical trapezoidal cross section and assumes steady uniform flow. The method is especially applicable to small streams because it accounts for transporting the bed material sediment discharge in the water above the bed, not the banks, and because it separates total hydraulic roughness into bed and bank components.

Theoretical Basis

The analytical method uses Brownlie's (1981)[1] resistance and sediment transport equations. These are regression equations based on about 1,000 records from 31 flume and field data sets. The data were carefully analyzed for accuracy and consistency by Brownlie. The resistance equations account for both grain and bed form roughness. The data covered a wide range of conditions as shown in the following tabulation:

Variable	Minimum	Maximum
d_{50}, mm	0.088	2.8
Unit discharge, cfs/ft	0.129	430
Discharge, cfs	0.11	706,000
Slope	0.000003	0.037
Hydraulic radius, ft	0.082	56
Temperature, °C	0	63

All of the data had width-to-depth ratios greater than 4, and the gradation coefficients of the bed material were equal to or less than 5.

Brownlie developed separate resistance equations for upper and lower regime flow. The equations are dimensionless and can be used with any consistent set of units.

Upper regime:

$$R_b = 0.2836\, d_{50}\, q_*^{0.6248}\, S^{-0.2877}\, \sigma^{0.0813} \tag{A1}$$

Lower regime:

$$R_b = 0.3742\, d_{50}\, q_*^{0.6539}\, S^{-0.2542}\, \sigma^{0.1050} \tag{A2}$$

where:

R_b = hydraulic radius associated with the bed

d_{50} = median grain size

$$q_* = \frac{VD}{\sqrt{g\, d_{50}^3}}$$

V = average velocity

[1] Brownlie, William R. 1981. "Prediction of flow depth and sediment discharge in open channels," Report No. KH-R-43A, California Institute of Technology, Pasadena, CA.

D = water depth

g = acceleration of gravity

S = slope

σ = geometric bed material gradation coefficient

To determine if upper or lower regime flow exists for a given set of hydraulic conditions, a grain Froude number F_g and a variable F_g' were defined by Brownlie. According to Brownlie, upper regime occurs if $S >$ 0.006 or if $F_g > 1.25F_g'$, and lower regime occurs if $F_g < 0.8F_g'$. Between these limits is the transition zone. In the analytical method, $F_g = F_g'$ is used to distinguish between upper and lower regime flow. The program will inform the user which regime is being assumed in the calculations and if the bed forms are in the transition zone.

$$F_g = \frac{V}{\sqrt{g\, d_{50} \left(\frac{\gamma_s - \gamma}{\gamma} \right)}} \tag{A3}$$

$$F_g' = \frac{1.74}{S^{.3333}}$$

where:

γ_s = specific weight of sediment

γ = specific weight of water

The hydraulic radius of the side slope is calculated using Manning's equation:

$$R_s = \left(\frac{V n_s}{1.486\, S^{0.5}} \right)^{1.5} \tag{A4}$$

where:

R_s = hydraulic radius associated with the side slopes, ft

V = average velocity, fps

n_s = Manning's roughness coefficient for the bank

If the roughness height k_s of the bank is known, then it can be used instead of Manning's roughness coefficient to define bank roughness. The model uses Strickler's equation to calculate the bank roughness coefficient:

$$n_s = 0.039\, k_s^{1/6} \tag{A5}$$

where k_s is the roughness height in feet. For riprap, k_s should be set equal to the minimum design D_{90}.

Composite hydraulic parameters are partitioned in the manner proposed by Einstein (1950):[1]

$$A = R_b P_b + R_s P_s \tag{A6}$$

where:

A = total cross-sectional area

P_b = perimeter of the bed

P_s = perimeter of the side slopes

This method assumes that the average velocity for the total cross section is representative of the average velocity in each subsection.

Concentration C in parts per meter is calculated using the Brownlie sediment transport equation, which is also a regression equation, and is based on the same extensive set of flume and field data used to develop his resistance equations. This equation was chosen because of its compatibility with the resistance equations, which are coupled with the sediment transport equation in the numerical solution:

$$C = 9022 \left(F_g - F_{go}\right)^{1.978} S^{0.6601} \left(\frac{R_b}{d_{50}}\right)^{-0.3301} \tag{A7}$$

$$F_{go} = \frac{4.596\, \tau_{*o}^{0.5293}}{S^{0.1405}\, \sigma^{0.1606}} \tag{A8}$$

$$\tau_{*o} = \frac{\gamma\, R_b\, S}{(\gamma_s - \gamma)\, d_{50}} \tag{A9}$$

A typical cross section, with the critical hydraulic parameters labeled, is shown in Figure A1. The concentration calculated from the sediment transport equation applies only vertically above the bed. Total sediment transport in weight per unit time is calculated by the following equation:

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1 Einstein, Hans A. (1950). "The bed load function for sediment transportation in open channels," Technical Bulletin 1026, U.S. Department of Agriculture, Soil Conservation Service, Washington, DC.

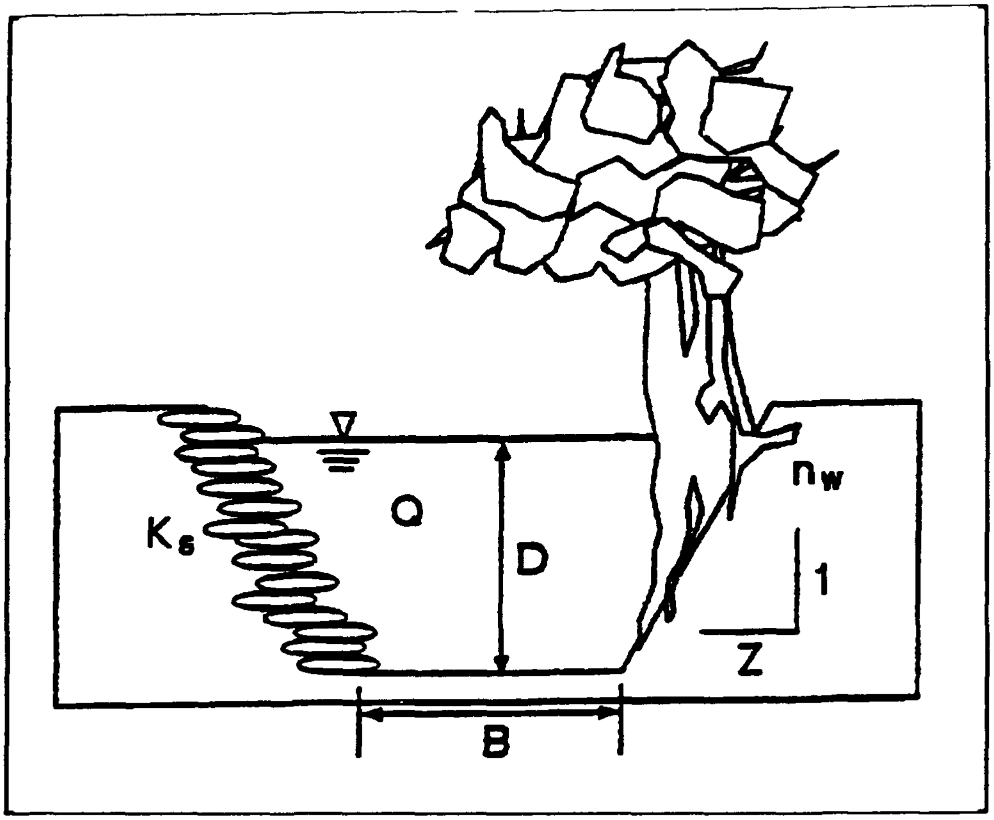

Figure A1. Typical cross section used in analytical method

$$Q_s = \gamma C B D V \tag{A10}$$

where:

Q_s = sediment transport in weight/time

B = base width

An average concentration for the total discharge is then calculated:

$$\overline{C} = \frac{Q_s}{0.0027\,Q} \tag{A11}$$

where:

$\overline{C}$ = concentration using the total discharge, ppm

Q_s = sediment transport, tons/day

Q = discharge, cfs

Using the Method

This method is coded in the computer program SAM, Hydraulic Design Package for Flood Control Channels. The first step in a stability analysis is to determine the bed material sediment load entering the project reach. This is calculated from a supply reach that has been reduced to a typical trapezoidal cross-section. Required input data are base width, side slope, bank roughness coefficient, bed material d_{50}, bed material gradation coefficient, average slope, and channel-forming water discharge. It is important that the base width is representative of the total movable-bed width of the channel. The bed roughness must include grain and form roughnesses. The bank roughness must include all roughness factors, (i.e., channel irregularities, variations of channel cross-section shape, relative effect of obstructions, vegetation, and sinuosity). Only flow that is vertically above the bed is considered capable of transporting the bed material sediment load.

The second step in the method is to develop a family of slope-width solutions that satisfy the resistance and sediment transport equations. For each combination of slope and base width, a unique value of depth is calculated. (This can be done to evaluate stability in an existing channel or in a proposed design channel.) If the existing channel is known to be stable or has a known aggradation or degradation history, then the supply reach characteristics can be adjusted to reproduce that known channel response. In addition to the discharge, sediment inflow, and bed composition from step 1, bank side slope and roughness for the project channel must be designated.

An example stability curve is shown in Figure A2. Any combination of slope and base width from this curve will be stable for the prescribed channel design discharge. Combinations of width and slope that plot above the stability curve will result in degradation, and combinations below the curve will result in aggradation. The greater the distance from the curve, the more severe the instability.

Constraints on this wide range of solutions may result from a maximum possible slope, or a width constraint due to right-of-way. Maximum allowable depth could also be a constraint. Depth is not plotted in Figure A2, but it is calculated for each slope and width combination determined. With constraints, the range of solutions is reduced.

Different water and sediment discharges will produce different stability curves. First, the stable channel solution is obtained for the channel-forming discharge. Then, stability curves are calculated for a range of discharges to determine how sensitive the channel dimensions are to variations in water and sediment inflow events. For example, Figure A3 shows that channel dimensions that are stable for the channel-forming discharge would degrade or aggrade during a flood event depending on the combination of width-slope solutions chosen for the design. That is, points 1, 2,

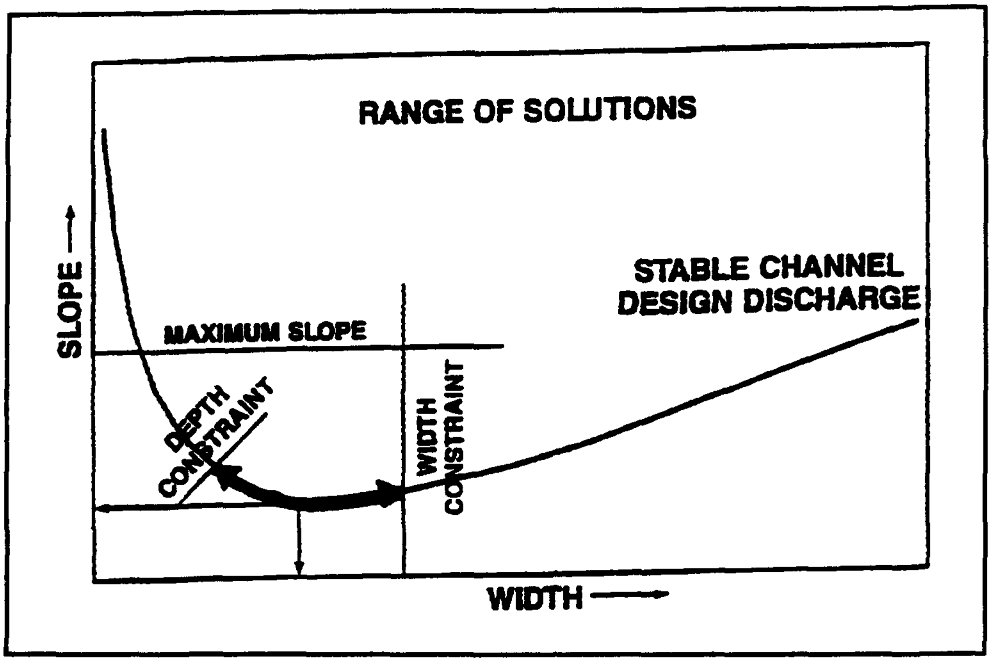

Figure A2. Range of solutions with project constraints

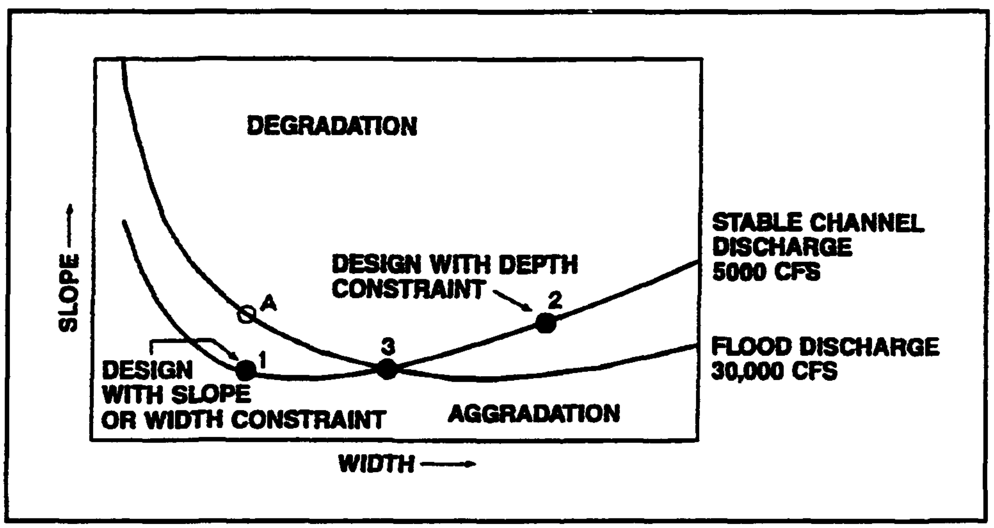

Figure A3. Stability curves showing channel tendencies during a flood discharge

and 3 represent possible combinations of slope and width that are stable for the design discharge of 5,000 cfs. However, the stability curve for a water discharge of 30,000 cfs, flowing through the same supply reach, plots above the channel-forming discharge curve on the left side of point 3 and below it on the right side. This indicates that a channel designed for point 1 could be expected to move toward a new stable width, depth, and slope somewhere between point A and point 3 during a discharge of 30,000 cfs. This could require increasing both width and slope. A

channel designed for point 2 would be expected to decrease either width and slope or both. It would be ideal to design a channel close to the geometry for the supply reach, that is, where the stability curves for all discharges intersect, location 3 on Figure A3.

There are options in the SAM program to assign a value for slope, thereby obtaining unique solutions for width and depth. Typically there will be two solutions for each slope.

The stable channel dimensions are calculated for a range of widths on either side of the regime value. That regime value is calculated using the hydraulic geometry equation proposed in EM 1110-2-1418.

$$B = 2.0\ Q^{0.5} \tag{A12}$$

where:

B = base width, ft

Q = discharge, cfs

The SAM program then assigns 20 base widths for the calculation, each with an increment of 0.1B. Calculations for these conditions are displayed as output. Stability curves can then be plotted from these data. The midpoint of the calculated base widths can be set as an option.

A solution for minimum stream power is also obtained in the model. This represents the minimum slope that will transport the incoming sediment load. Solution for minimum slope is obtained by using a second-order Lagrangian interpolation scheme. Opinions are divided regarding the use of minimum stream power to uniquely define channel stability.

Conclusions

The analytical method presented herein is intended for use in estimating channel dimensions for preliminary design studies. It accounts for the movement of sediment and for varying roughness due to changing bed forms. It also attempts to account for the effects of bank roughness that can be significant in small channels and are essentially neglected in many approaches that assume wide channels.

Simple techniques, such as the stable channel analytical method, require much more engineering judgment to apply successfully than more complex methods. A thorough knowledge of the stream is essential for a reliable stable channel analysis.

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13. ABSTRACT *(Maximum 200 words)*

The analytical method presented herein, developed through the Flood Control Channels Research Program, is intended for use to estimate channel dimensions for preliminary design studies. It accounts for the movement of sediment and for varying roughness due to changing bed forms. It also attempts to account for the effects of bank roughness that can be significant in small channels and is essentially neglected in many approaches that assume wide channels. This analytical approach determines dependent design variables of width, slope, and depth from the independent variables of discharge, sediment inflow and bed material composition. it is especially applicable to small streams because it accounts for transporting the bed material sediment discharge in the water above the bed, not the banks, and because it separates total hydraulic roughness into bed and bank components.

A stability analysis can be done by first determining the bed material sediment load entering the project reach, and then analyzing the family of slope-width solutions that satisfy the resistance and sediment transport equations. The resulting stability curve provides many combinations of slope and base width, all of which will be stable for the prescribed channel design discharge. Combinations of width and slope that plot above the stability curve will result in degradation, and combinations below the curve will result in aggradation. The greater the distance from the curve, the more severe the instability.

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This method is coded in the computer program SAM, "Hydraulic Design Package for Flood Control Channels." Two case studies illustrating this method are also presented here. However, simple techniques, such as the stable channel analytical method, require much more engineering judgment to apply successfully than more complex methods. A thorough knowledge of the stream is essential for a reliable stable channel analysis.